T. Buck Construction, Inc.

249 Merrow Road, Auburn, Maine 04210-8319

(207) 783-6223 * (FAX) 783-3970

TRAFFIC CONTROL PLAN

Vermont Agency of Transportation

Bridge Replacement in town of Burke, VT
BRF 0269(13)

Submitted 3/30/15
REVISED 4/20/15

STATE OF VERMONT
ERIC A. CALDERWOOD
NO. 8711
Structural
LICENSED
PROFESSIONAL ENGINEER

A. DESCRIPTION OF PROJECT:

This project involves the removal of bridge 13 and portions of its abutments and foundation. Bridge 13 will be replaced with a precast structure, spanning 56 Feet over Dish Mill Brook, on new footings along the same alignment. Bridge 13 is located in the town of Burke, on VT Route 114, approximately .47 miles easterly of the Lyndon/Burke Town line. The width of the bridge will be increased to 36 feet – 10 inches.

This plan will illustrate T Buck's intensions on how to safely move traffic and pedestrians through or around the project site at each stage of the process. In General, T Buck will attempt to minimize the impacts to the traveling public by channelizing them around the work and away from the project as the contract specifications describe.

B. KEY PERSONNNEL CONTRACT INFO

- Site Superintendent, T Buck Construction:
  - Harry Pottle — 207-754-2169
harry@tbuckcon.net
- Project Manager, T Buck Construction:
  - Brian Emmons — 207-212-0960 (cell)
207-783-6223 x 205 (office)
brian@tbuckcon.net
- Resident Engineer, Vermont Agency of Transportation:
  - Kevin McClure — 802-917-4624 (cell)
TBD (Field office)
Kevin.McClure@state.vt.us
- Lyndonville, VT Police Station
  - Non-Emergency — 802-626-1721
  - Emergency — 9-1-1
- Burke, VT Volunteer Fire Brigade
  - Non-Emergency — 802-626-5484
  - Emergency — 9-1-1
- Lyndonville Rescue
  - Non-Emergency — 802-626-1101
  - Emergency — 9-1-1

Note: other individuals may be added as necessary.

C. SCHEDULE OF PHASING

This project will be divided into three phases: Pre-closure, Closure, and Post Closure. The phases are described below and shown on the applicable sketches in the appendix of this plan.

- Pre-closure: Mid April – May 25th

Description	Start	Stop
Project Survey / Layout	4/20	4/24
Installation of erosion control devices	4/20	4/23
Installation of traffic control devices	4/20	4/24
Installation of temporary bridge	4/27	5/1
Pre-excavation of abutment 2 piles	5/4	5/8
Installation of abutment 1 piles	5/4	5/9
Installation abutment 2 piles	5/11	5/16

- Closure: May 26th – June 15th

  See Critical Path Method Schedule (spec section 900.645)

- Post-closure: June 15th – August 22nd

Description	Start	Stop
Form and place retaining wall footing	6/15	6/26
Form and place retaining wall	6/29	7/3
Form and place sidewalk (on bridge)	6/15	6/19
Form and place Northern Texas rail	6/22	6/26
Form and place Southern Texas rail	6/29	7/3
Form and place C.I.P. sidewalks	7/6	7/17
Install guardrail and approach rail	7/20	7/24
Remove temp traffic barrier	7/24	7/24
Final Pavement	7/27	7/30

D. EXPLANTION OF PHASING

Pre-closure:

During this phase, traffic will be maintained in one alternating lane controlled by flaggers during daytime hours and maintained on the existing two lane alignment at night or whenever possible.

The construction activities related to traffic during this phase will be limited to day time hours and will consist of installing the temporary traffic bridge and the pre-excavation of abutment 2 piles and pile installation. The contract requires a minimum of 4 of the 6 abutment 2 piles be installed prior to the bridge closure period. The contract also states that daily lane closures for the purposes of pre-excavating and installing abutment 2 piles can occur for a maximum of 2 weeks leading up to the bridge closure.

T Buck plans to install a one lane temporary bridge immediately upstream of the existing bridge to accommodate the single lane of traffic during the day time. The sketch and calculations in appendix B of this plan depict the location and details for the temporary bridge. T Buck will need brief sporadic lane closures during the installation of the temporary bridge before the pile work begins. Brief sporadic lane closures are defined to be short in duration (2-3 hours) and only when necessary (i.e. during equipment moves, material delivery, and launching of temporary)

During this phase pedestrians will either utilize the existing bridge (night time) or the temporary bridge (day time). Flaggers will control the jobsite during day.

The entire project will be ADA compliant at all times and during all phases of this project

The intent is to have all traffic be controlled by flaggers. Essentially all traffic will stop or move through the jobsite very slowly while the one lane temporary is active. Pedestrians and bicycles will be given a safe opportunity to cross the bridge as well.

Emergency vehicles will be given immediate precedence to move through the site at all times. And DMV will be notified of any / all restrictions through or around the jobsite.

Closure Period:

During this phase, all vehicle traffic will be detoured around the project site while the bridge is replaced. The detour plan is given in the contract documents and can be seen on plan sheet(s) 21-23. Those applicable contract sheets can be seen in appendix B of this plan. A detailed description of the vehicle detour can be seen in section G of this plan

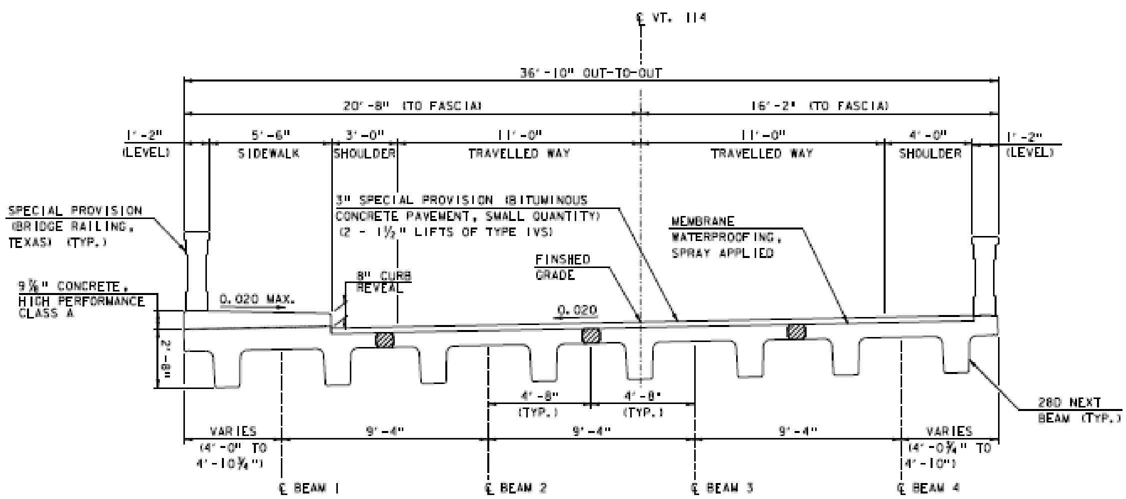
During this phase pedestrians and bicycles will be able to utilize a temporary pedestrian bridge. The bridge will be located upstream of where the temporary bridge was during the pre-closure phase. Temporary ramps will be constructed up to and off of the bridge to minimize any unnecessary earth disturbance outside of the construction limits. T Buck has obtained permission from abutting land owners for use of a walkway during the bridge closure. Typical land use agreements will be available upon request. Vtrans Off-site Activity submittals have been reviewed and approved. The temporary approaches will consist of gravel placed on filter fabric to allow for a clean separation.

<u>Post Closure:</u>

This phase will begin with the opening of the new bridge. T Buck intends to open the bridge to traffic using temporary traffic barrier to delineate 2 lanes 11’ wide as described in the contract documents. The typical section can be seen in the picture below. During daytime hours, traffic may be reduced to one lane during concrete placements, material delivery, subcontractor work, etc. each night, the bridge will be opened to the 2 specified 11’ lanes.

The construction activities during the phase will include the forming & placing of the sidewalk(s) and Texas style bridge railing. It will also include the forming and placing of the retaining wall for the bed & breakfast located near station 26+50 (LT). Once the guardrail and approach railing is installed, the temporary traffic barrier will be removed. As soon as the project is relatively complete in terms of curbing, railings, etc, the final lift(s) of pavement will be installed full width.

Pedestrians will use the new bridge to move through the project limits and when the barrier is removed, the new permanent sidewalks will be available for use.



TYPICAL BRIDGE SECTION

SCALE: ⅜" = 1'-0"

E. TEMPORARY TRAFFIC CONTROL DEVICES

Site Specific

The temporary traffic control devices will be installed in accordance with the current version of the MUTCD and will generally consist of the following

- Typical Approach Signing
- Temporary Traffic Barrier (Jersey barrier)
- Type III Barricades
- Traffic Drums and/or Cones

Detour

The detour signing will be layout in accordance with the plans and specifications. The resident engineer will in involved with the layout and approve any/all changes and locations of each sign assembly

Portable Changeable Message signs will be placed in the general location shown on the plans. Again, the resident engineer will have final approval of locations. The signs will switch between two screens. And the messages can be seen on sheet 23 of the contract drawings. The message may be changed if requested by the resident engineer

Miscellaneous

The site superintendent will have an inventory of basic signs to replace any that are broken or damaged throughout the project. In conjunction with the resident engineer, we will work to fabricate and install any new or applicable signs or devices that are necessary after the typical devices are installed. Ultimately we will work with the town and all officials to make sure the traveling public is moved through or around our project safely and efficiently.

F. FLAGGING

The flagging subcontractor that will be utilized on this project is ADA Traffic Control and is located in Bridgewater, VT. Daily and/or weekly slips will be turned into the resident engineer for payment under item 630.15. In general a discussion will take place with the resident before the flaggers are scheduled so that all interested parties will be aware and may comment prior to implementing flagging operations.

The flagging operations should be limited to the pre-closure and post-closure periods.

Flaggers will be equipped with radios and will be able to communicate with all flaggers to properly direct traffic through the project site.

G. PECIAL DETOURS

Onsite Temporary Bridge:

The temporary bridge will be installed so to keep traffic away from pile driving operations during the pre-closure phase. When the temporary bridge is no longer needed for traffic, it will be relocated upstream of the bridge and utilized as a pedestrian bridge during the closure period.

During the closure, traffic will be maintained on a regional detour via routes VT114, VT 105, VT 5A, VT 16 and US 5 between East Burke, Brighton, Charleston, Westmore, and Lyndon. Interstate 91 between exits 23 and 25 will also be used. The off-site vehicle detour is shown in great detail on plan sheets 21-22 of the contract drawings. For reference, those sheets are included in this plan in Appendix B

H. NIGHT WORKING PLAN

There is no night work anticipated during the pre-closure period.

During the closure period, all night work will be performed in accordance with local and state regulations including adhering to the approved lighting plan.

There is no night work anticipated in the post-closure period.

APPENDIX A

SIGN SCHEDULE

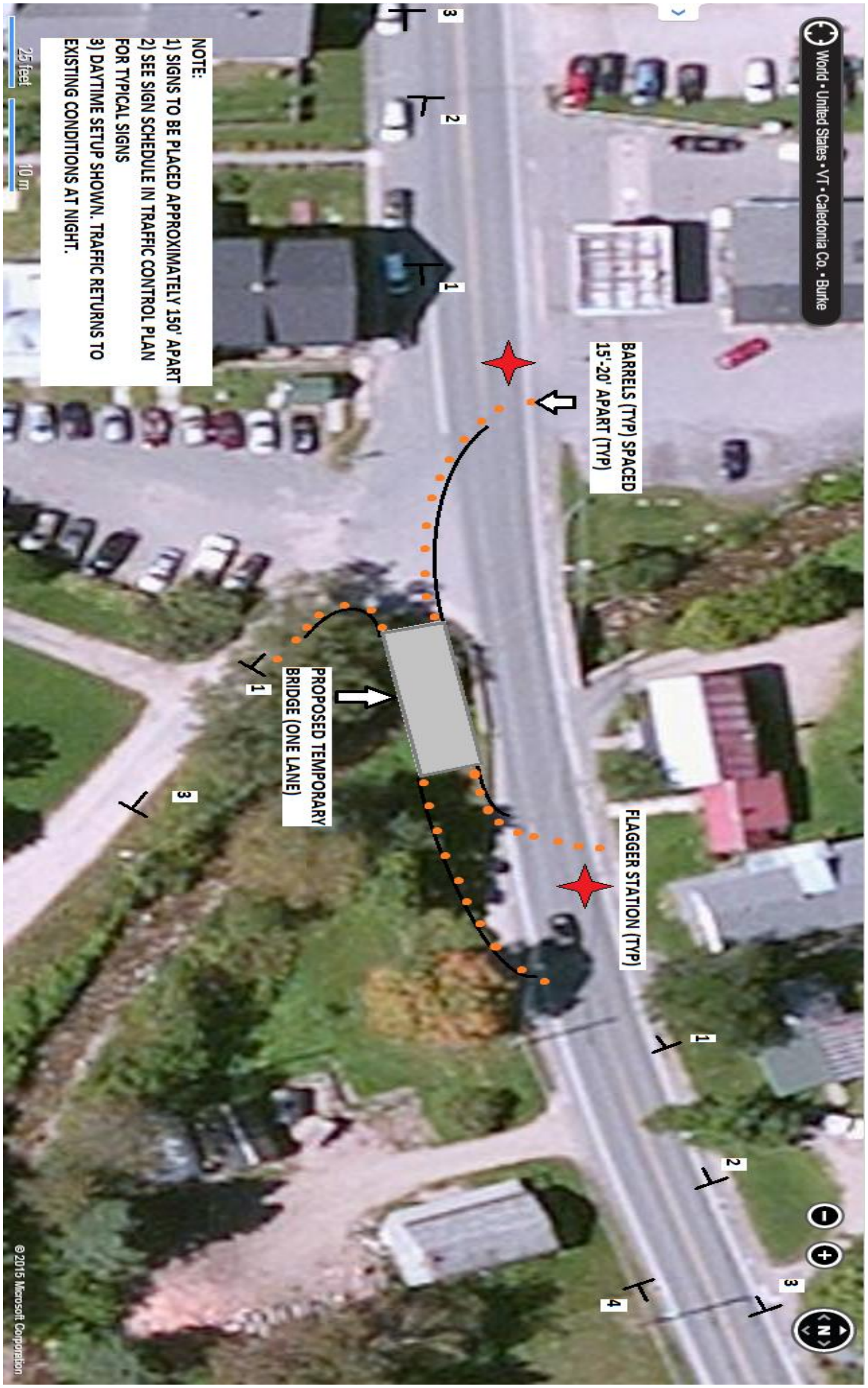
IMAGE	MUTCD ID NO.	QUANT.	SIZE	TEXT	LOCATION ON PROPOSED CONDITIONS PLAN
	W20-1	3	48"X48"	ROAD WORK AHEAD	3
	W20-4	2	48"X48"	ONE LANE ROAD AHEAD	2
	G20-2A	2	48"X24"	END ROAD WORK	4
	W20-7A	2	48"X48"	FLAGGER SYMBOL	1
	W3-4	2	48 x 48	BE PREPARED TO STOP	NONE, SIGN WILL BE IN INVENTORY IN CASE OF EMERGENCY
	W8-1	2	48"X48"	BUMP	NONE, SIGN MAY BE USED AFTER CLOSURE PERIOD
	M4-9AR	2		PEDESTRIAN DETOUR	
				SIDEWALK CLOSED CROSS HERE	
	W20-3			ROAD CLOSED AHEAD	

NOTE: 1. TYPICAL SIGNGS SHOWN. Other signs may be used as needed or as directed by the Resident Engineer.
NOTE: 2. Detour sign package included in Appendix B – Plan and Details (see sheet 23 of contract drawings.)

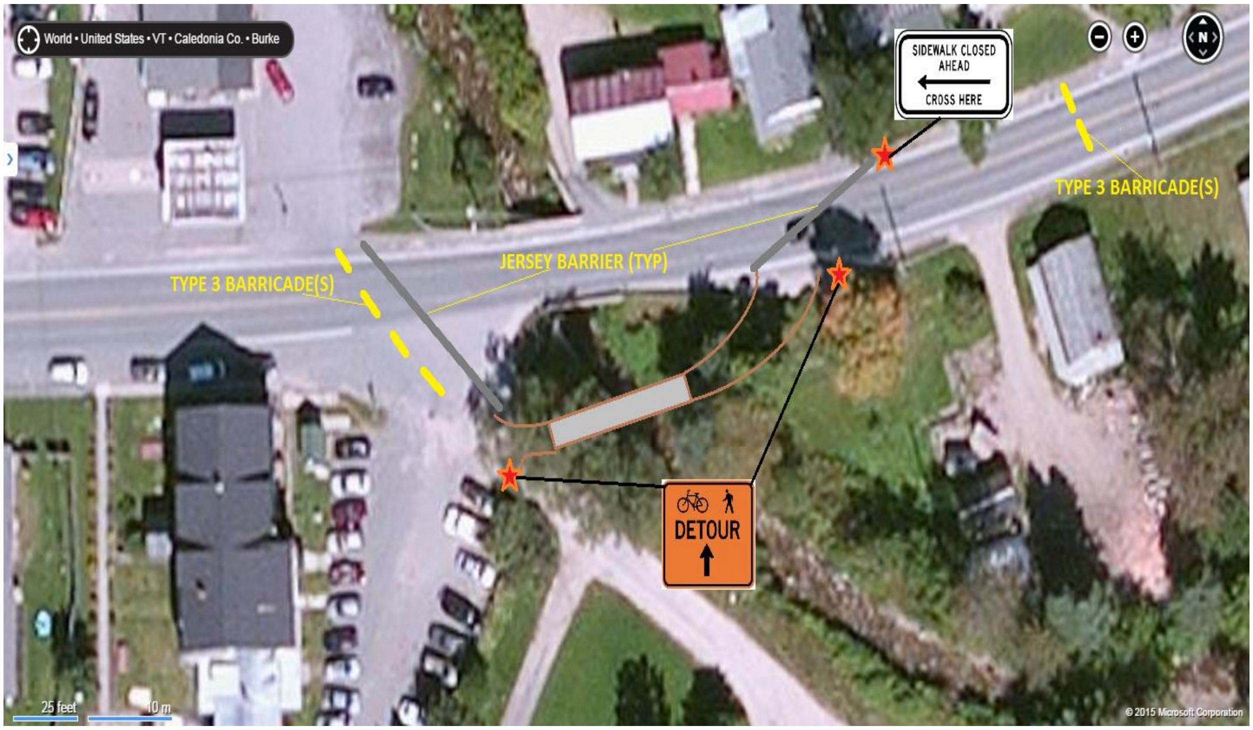
APENDIX B

PLANS AND DETAILS

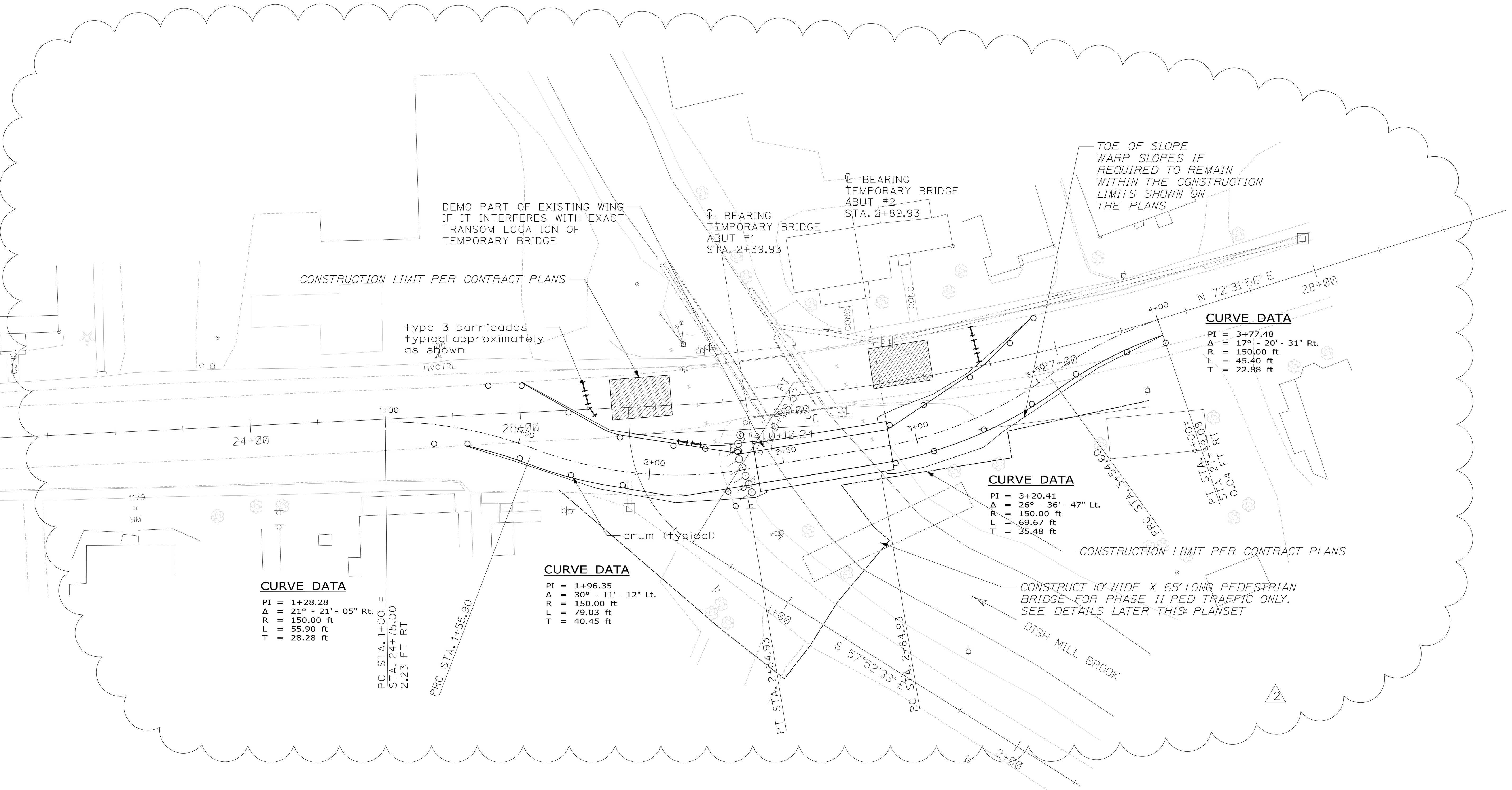
EXISTING CONDITIONS

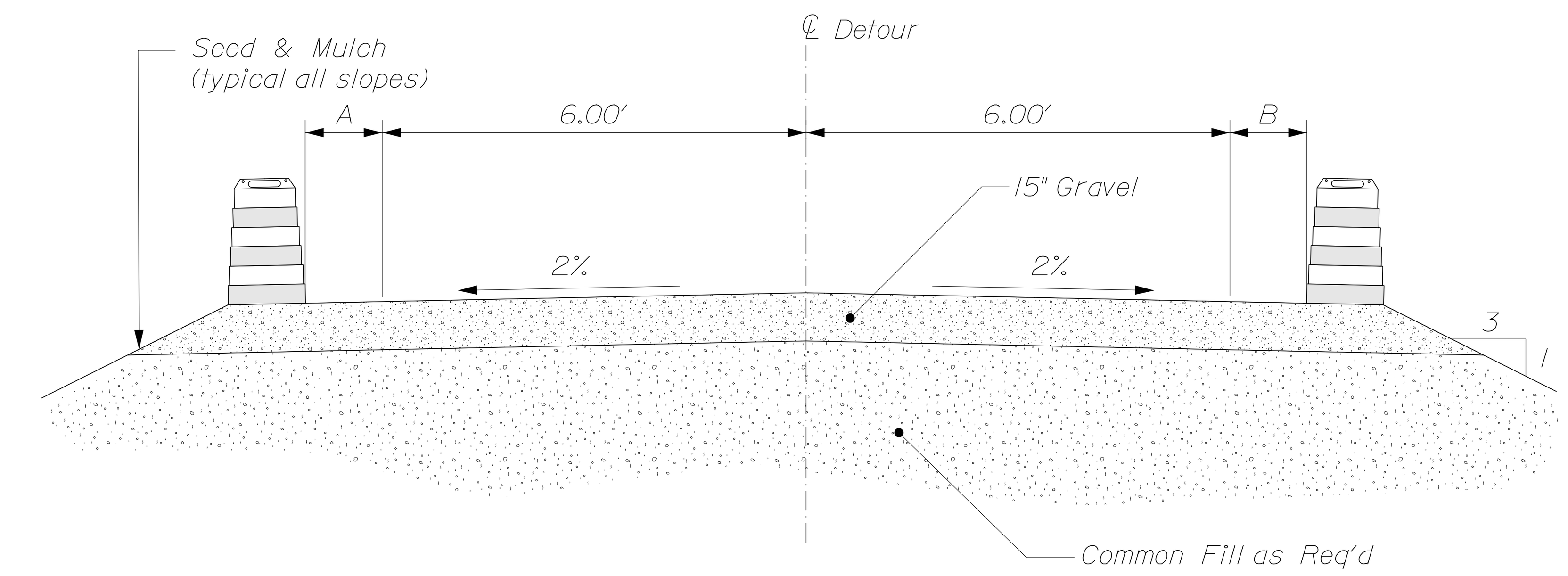


PRE-CLOSURE PERIOD



CLOSURE PERIOD

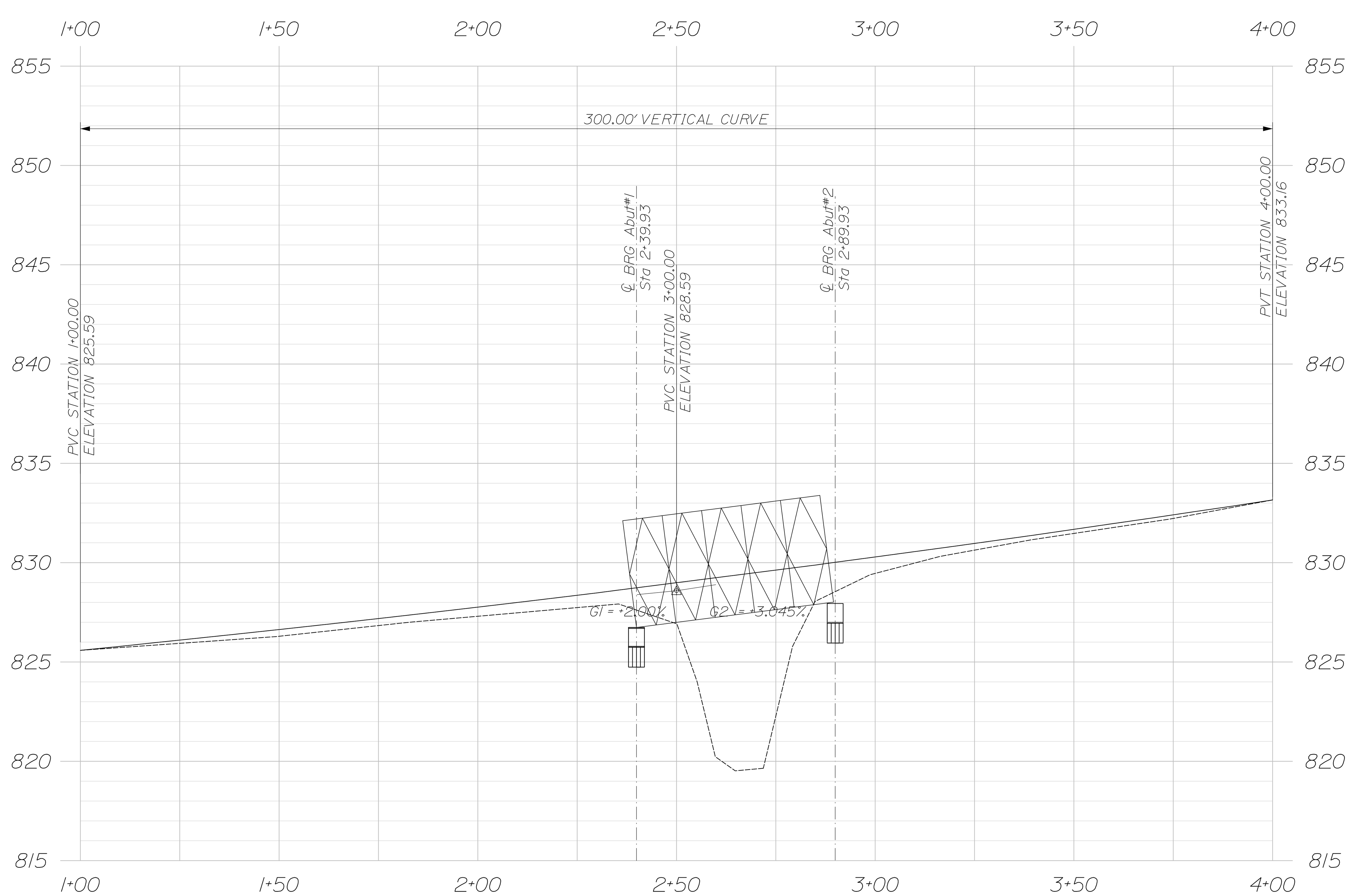




TYPICAL TEMPORARY APPROACH SECTION

(not to scale)
Dimension A- Additional Width provided on the Left for off tracking vehicles
Dimension B- Additional Width provided on the Right for off tracking vehicles

STA.	DIM A	DIM B
1+00	n/a	n/a
1+50	12.0'	0.0'
2+00	6.5'	1.0'
2+50	0.0'	0.0'
3+00	5.0'	0.0'
3+50	12.0'	0.0'
4+00	n/a	n/a



A/1 TEMPORARY PROFILE
HORIZONTAL: 1"=20'-0"
VERTICAL: 1"=4'-0"

GENERAL NOTES

1. THESE PLANS ARE NOT INTENDED TO BE USED ALONE BUT ARE INTENDED TO BE WORKED WITH THE CONTRACT DOCUMENTS FOR THE BRIDGE IN QUESTION VTRANS PROJECT NUMBER BRF-0269(13)
2. STEEL GRADE 50 KSI YIELD MIN. IN NEW OR GOOD USED CONDITION FOR DISTRIBUTION BEAMS - STEEL AS PROVIDED BY ACROW FOR ACROW 300 BRIDGE

   PEDESTRIAN BRIDGE STEEL TO BE ASTM A7 GRADE 33
3. REPORT ANY OBSERVED DISCREPANCY BETWEEN ACTUAL FIELD CONDITIONS AND THESE PLANS TO THE TEMPORARY BRIDGE ENGINEER OF RECORD IMMEDIATELY
4. DO NOT PROCEED WITH ANY DEPENDENT WORK UNTIL ANY SUCH REPORTED DISCREPANCY IS ADDRESSED TO THE SATISFACTION OF THE TEMPORARY BRIDGE ENGINEER OF RECORD.
5. FOR ABUTMENT NOTES REFER TO PLAN SHEET #2
6. CONSTRUCT TEMPORARY APPROACH FILLS FOR PEDESTRIAN BRIDGE SUCH THAT THEY ARE ADA COMPLIANT. CONSTRUCT THEM OF GRANULAR FILL ON TOP OF GEOTEXTILE. INSTALL SILT FENCING AS REQUIRED ON EACH OF THE 4 CORNERS OF THE BRIDGE APPROACHES. TIE SILT FENCING INTO TEMPORARY BACKWALL AT PED BRIDGE.

CALDERWOOD ENGINEERING, ETC.
STRUCTURAL ENGINEERING • DETAILING SERVICES
222 RIVER RD, RICHMOND, ME 04357 PH/FX (207)737-2007/(207) 737-2008

PREPARED FOR:
TBUCK CONSTRUCTION, INC
CEE 032-BR-15

P.E. NUMBER
DATE:

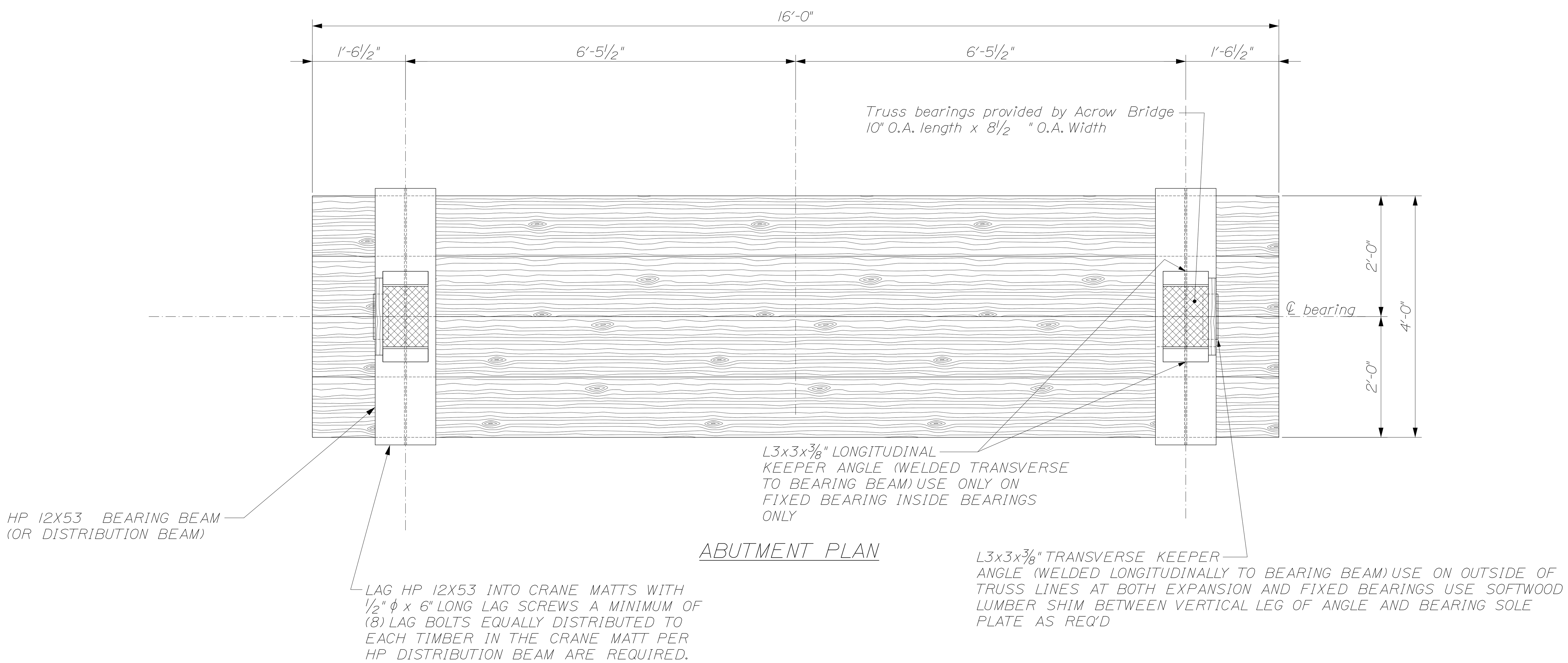
	BY	DATE
DESIGN-DETAILED	ETC	2015
CHECKED-REVIEWED		
REVISIONS 1 ROW CONST LIMIT ADDED		2015
REVISIONS 2 PED BRIDGE & MISC TEXT		2015
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BURKE, VT RTE 114 OVER DISH MILL BROOK

TEMPORARY DETOUR PLAN & PROFILE GENERAL NOTES

SHEET NUMBER

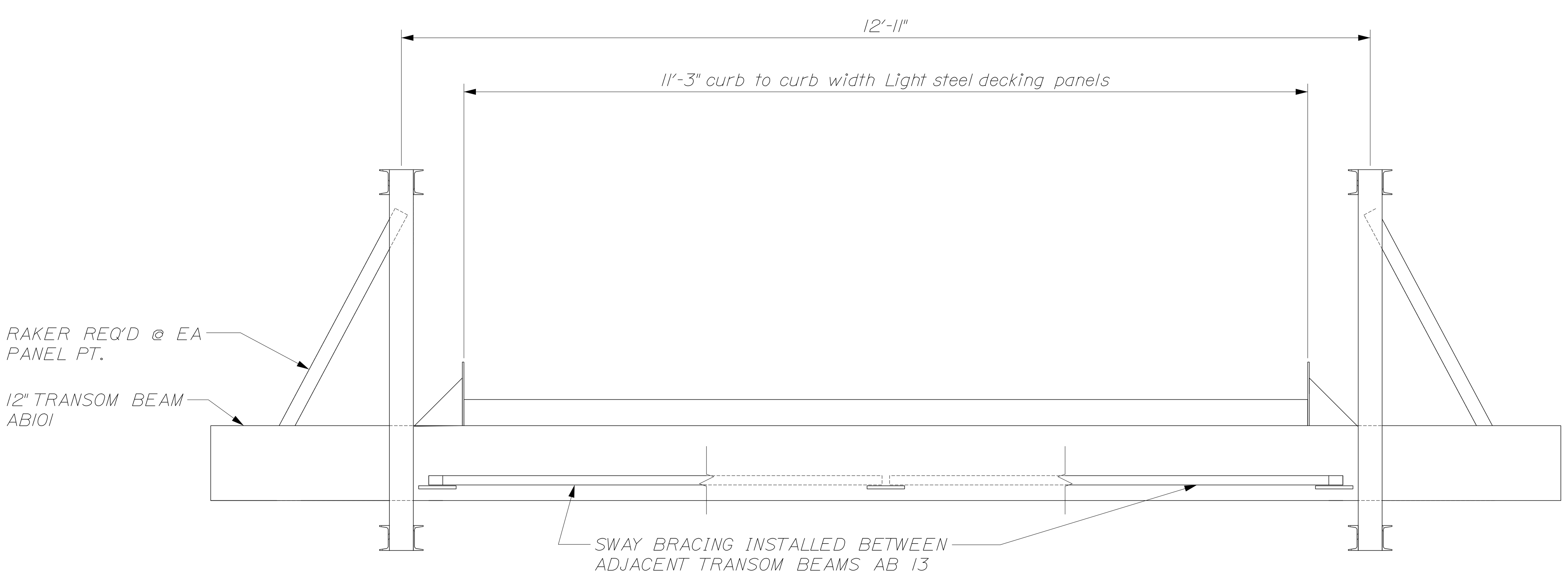
1



ABUTMENT PLAN

TOP OF DISTRIBUTION BEAM ELEVATIONS:
ABUTMENT #1 - 826.69
ABUTMENT #2 - 827.97

(vertical curve is approximated at each abutment)



2/2 DIMENSIONAL SECTION
¾"=1'-0"

GENERAL NOTES

1. STEEL HP SECTIONS SHALL BE IN NEW OR GOOD USED CONDITION

2. TIMBER BACKWALL SHALL BE CONSTRUCTED OF 8X8 TIMBERS USE SPF #2 OR EASTERN HEMLOCK #2 AT THE CONTRACTORS OPTION. BACKWALL MAY BEAR AT ENDS OF TRUSSES, EXACT DETAILS TO BE DETERMINED IN THE FIELD.

3. CRANE MATTS SHALL BE INSTALLED LEVEL ON UNDISTURBED NATIVE SOIL OR ON FULLY COMPACTED GRANULAR BORROW. CRUSHED STONE MAY BE USED IN LIEU OF GRANULAR BORROW, BUT ANY FILL MATERIALS SHALL BE FULLY COMPACTED IN ACCORDANCE WITH INDUSTRY STANDARD OF PRACTICE FOR TEMPORARY BRIDGE ABUTMENTS.

4. CRANE MATTS SHALL BE SOUND MATERIAL EITHER NEW OR GOOD USED CONDITION AND SHALL BE MIXED HARDWOOD, MIXED MAPLE OR MIXED OAK. CRANE MATTS SHALL BE THROUGH BOLTED AND SHALL BE A MINIMUM OF 12" THICK.

5. REPORT ANY OBSERVED DISCREPANCY BETWEEN THESE PLANS AND ACTUAL OBSERVED FIELD CONDITIONS TO THE TEMPORARY BRIDGE ENGINEER OF RECORD IMMEDIATELY.

6. DO NOT PROCEED WITH ANY DEPENDENT WORK UNTIL ANY SUCH REPORTED DISCREPANCY HAS BEEN RESOLVED TO THE SATISFACTION OF THE TEMPORARY BRIDGE ENGINEER OF RECORD.

7. THESE PLANS ARE NOT MEANT TO BE USED ALONE, BUT ARE TO BE WORKED IN CONJUNCTION WITH THE CONTRACT PLANS FOR THE HELMS BRIDGE REPLACEMENT.

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PREPARED FOR:
TBUCK CONSTRUCTION, INC

CEE 032-BR-15

P.E. NUMBER

DATE:

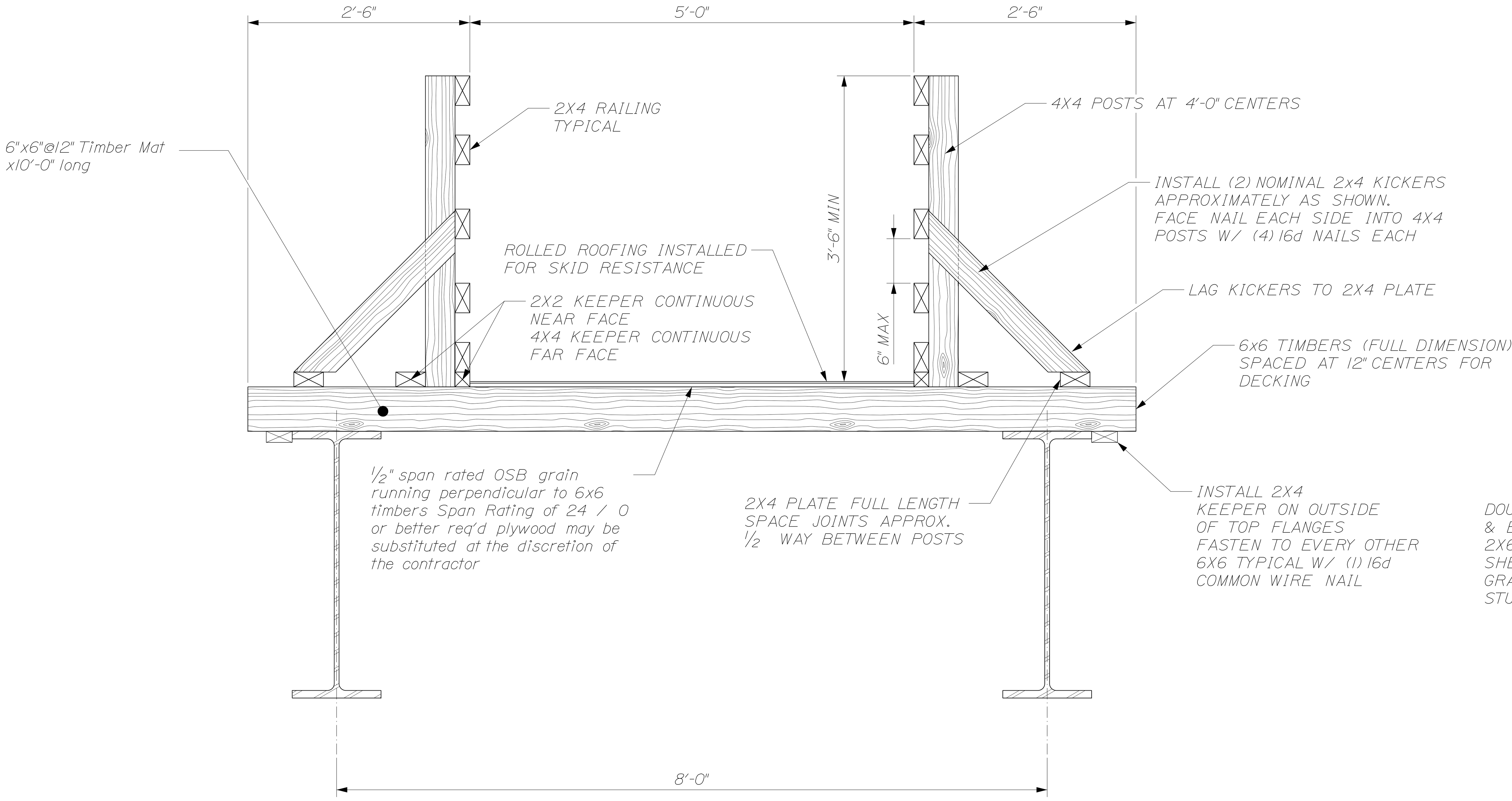
	BY	DATE
DESIGN-DETAILED	ETC	2015
CHECKED-REVIEWED		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BURKE, VT RTE 114
OVER DISH MILL BROOK

TEMPORARY BRIDGE ABUTMENTS AND SECTION

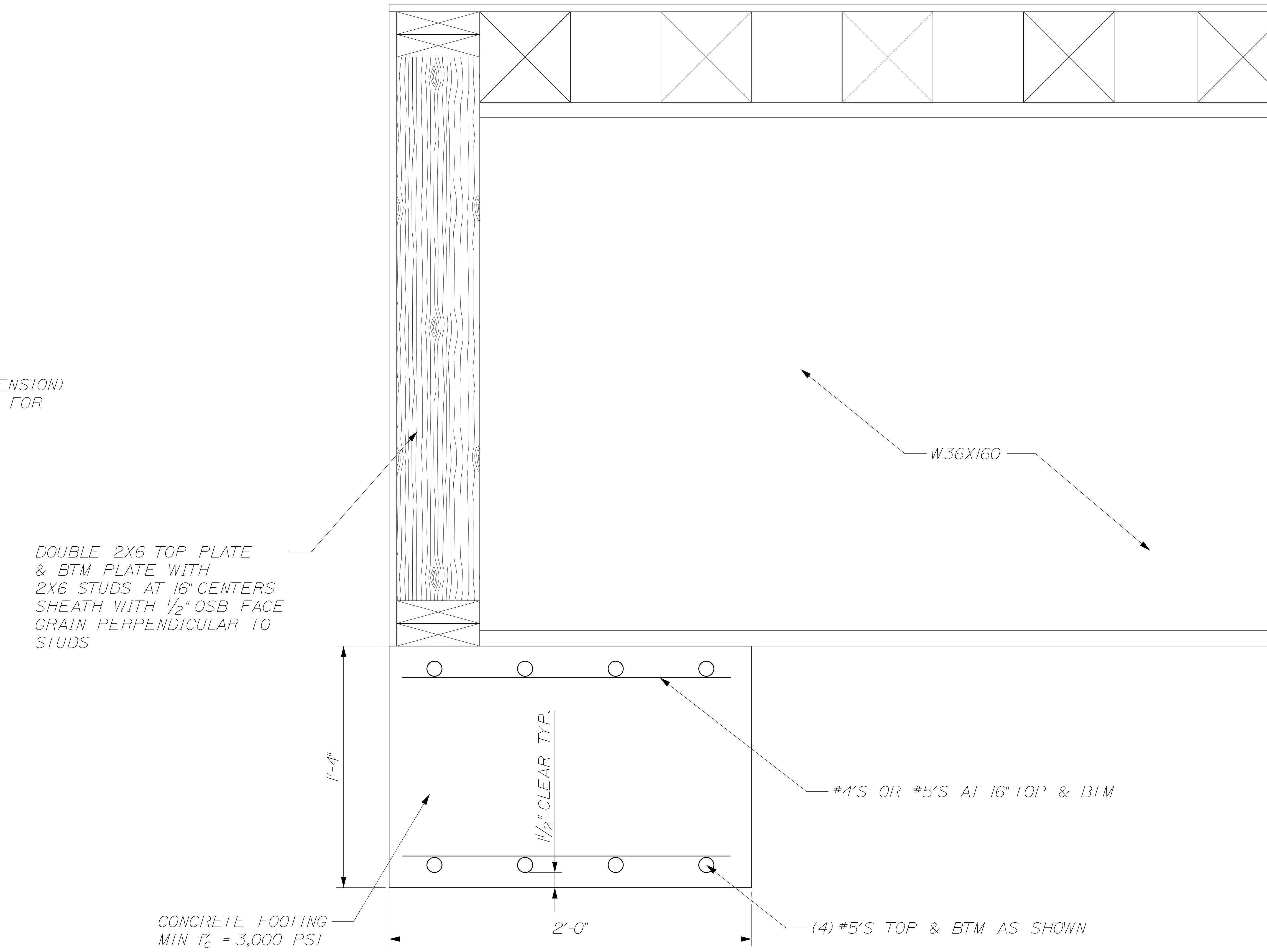
SHEET NUMBER

2



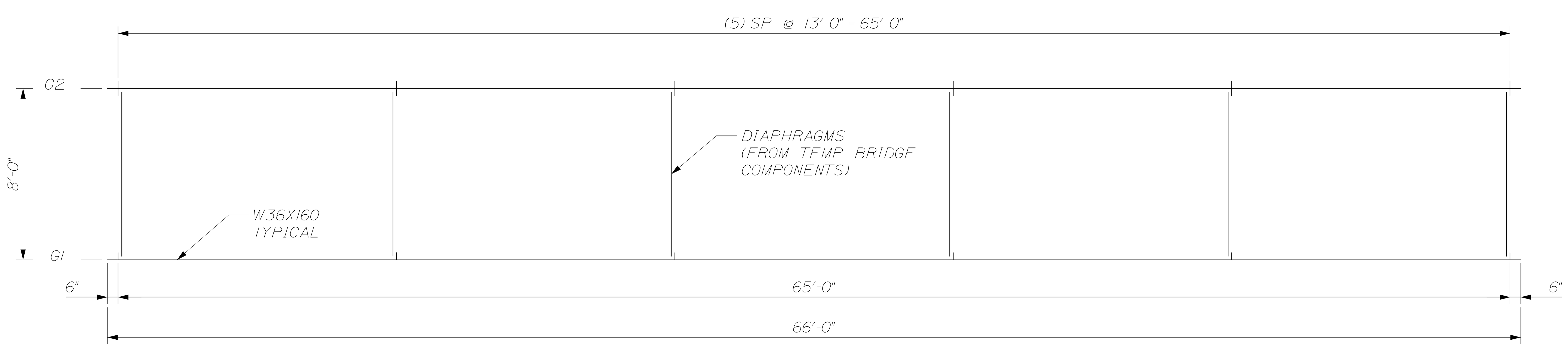
65' Span Ped Bridge Typical Section

1" = 1'-0"



ABUTMENT SECTION

CONSTRUCT 24" WIDE X 16" MIN DEEP ABUTMENT FOOTING 10' LONG. CENTER BRIDGE STRUCTURE ON ABUTMENT FOOTINGS



FRAMING PLAN -SPAN 1

1" = 4'-0"

GENERAL NOTES

1. ALL TIMBER SHOWN TO BE SPF #2 OR BETTER
2. EXACT NAILING SCHEDULE NOT SHOWN. FASTEN IN GENERAL CONFORMANCE TO INDUSTRY STANDARD PRACTICE
3. OSB TO BE SPAN RATED 24/0 TYPICAL 1/2" (ANY HIGHER SPAN RATING MAY BE SUBSTITUTED AT THE CONTRACTORS OPTION. AS MAY PLYWOOD)
4. ALL REBAR TO BE GRADE 60 PLAIN FINISH DEFORMED MEETING THE REQUIREMENTS OF ASTM A615

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PREPARED FOR:
TBUCK CONSTRUCTION, INC

CEE 032-BR-15

P.E. NUMBER

DATE:

	BY	DATE
DESIGN-DETAILED	ETC	2015
CHECKED-REVIEWED		
REVISIONS 1		
REVISIONS 2	SHEET ADDED	2015
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BURKE, VT RTE 114
OVER DISH MILL BROOK

PEDESTRIAN BRIDGE DETAILS

SHEET NUMBER

3

Temporary Bridge -
Rte 114 over Dishmill Brook
Supporting Calculations

In the Town of

Burke

032-br-15

Prepared for:

TBuck Construction Inc

By:

Calderwood Engineering etc

March, 2015

STATE OF VERMONT
ERIC T. CALDERWOOD
NO. 8711
Structural
LICENSED
PROFESSIONAL ENGINEER

Photo: US Rte 1 over the Mousam River in Kennebunk Me (Temporary Bridge
Courtesy of TBuck Construction, Inc & MaineDOT

CHAPTER EIGHT – CONSTRUCTION

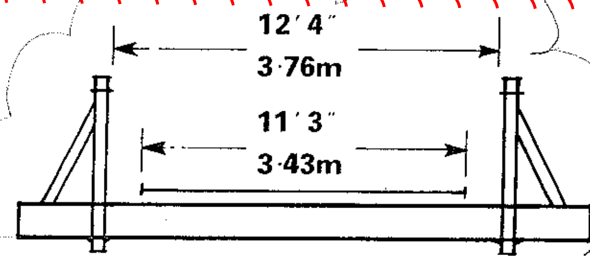
GENERAL

Acrow Panel Bridge is supplied in four roadway widths.

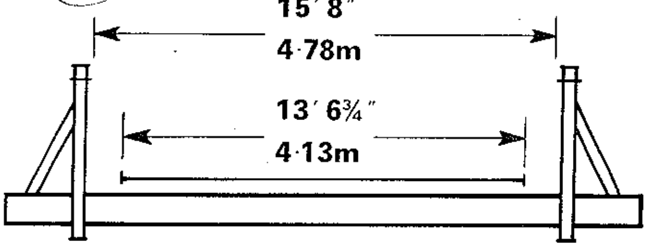
Standard	}	Single Lane.
Extra Wide	}	Single Lane.
Ultra Wide	}	Single Lane.
Double Wide		Two Lane.

Roadway widths and clearances are shown below.

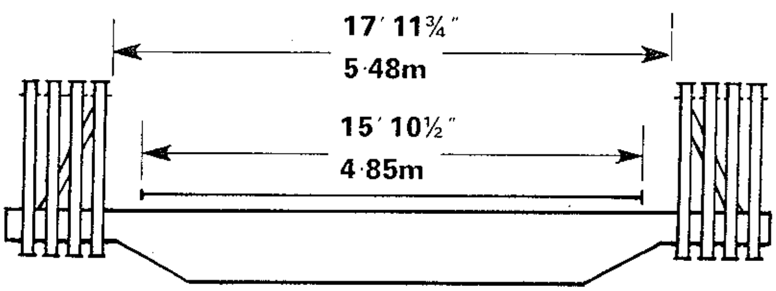
TRANSOM
12"
DECK 4 3/16



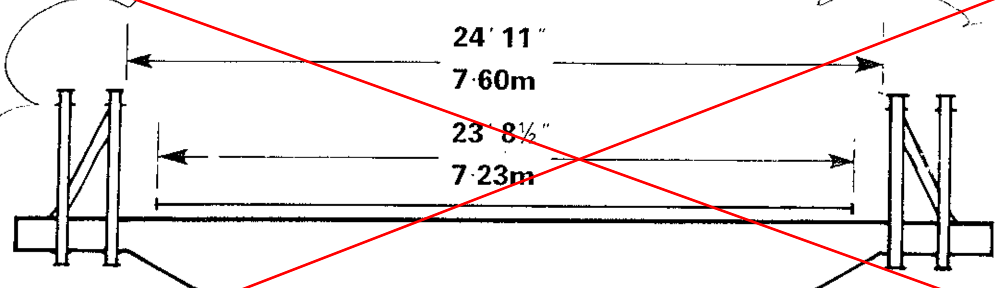
STANDARD



EXTRA WIDE



ULTRA WIDE



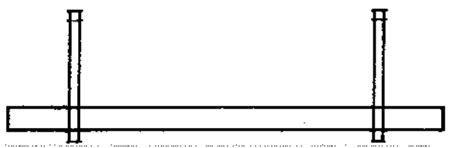
DOUBLE WIDE

Standard, Extra Wide and Double Wide bridges are for Highway use and are supplied with either light or heavy decking to suit various Highway Loading Specifications.

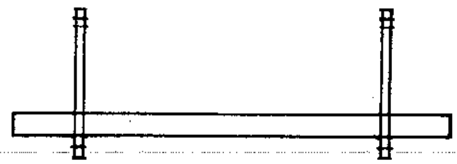
Extra Wide bridges can also be supplied with Super Heavy decking where very heavy axle and wheel loadings are involved.

The Ultra Wide bridge is specifically designed for the latest types of heavy earth moving plant and is therefore usually only supplied with Super Heavy decking.

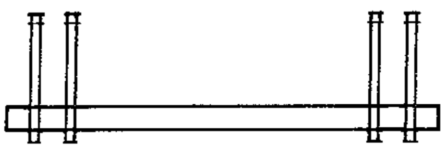
Single lane Highway bridges can be built in all available forms of construction, which are as follows:—



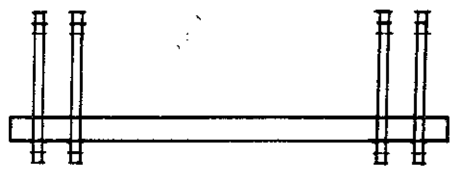
SINGLE SINGLE



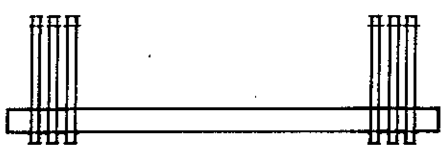
SINGLE SINGLE REINFORCED



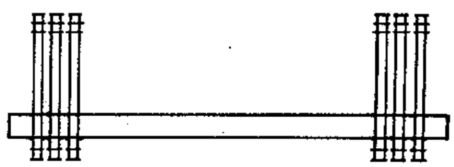
DOUBLE SINGLE



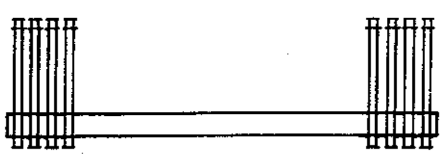
DOUBLE SINGLE REINFORCED



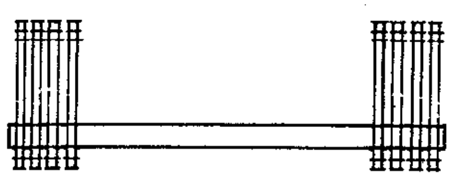
TRIPLE SINGLE



TRIPLE SINGLE REINFORCED



QUADRUPLE SINGLE



QUADRUPLE SINGLE REINFORCED

TABLE 7 – Continued
WEIGHTS OF BRIDGES (long tons)

		TWO END BAYS	ONE INTERNAL BAY	EXTENSIONS
TRUSSES ONLY	SS	2.30	0.58	–
	SSR	2.50	0.98	–
	DS	3.99	1.18	–
	DSR	4.39	1.98	–
	TS	5.47	1.74	–
	TSR	6.07	2.94	–
	QS	7.12	2.30	–
	QSR	7.92	3.90	–
	DD	6.39	2.36	–
	DDR	6.79	3.16	–
	TD	9.08	3.48	–
	TDR	9.68	4.68	–
	QD	11.83	4.62	–
	QDR	12.63	6.22	–
STD LS		2.76 ✓	1.62 ✓	1.05
STD LT		2.48	1.44	0.85
STD LTw/oT		2.08	1.17	0.58
STD HS		3.32	1.94	1.20
STD HT		3.39	2.00	1.24
STD HTw/oT		2.60	1.47	0.72
EW LS		3.36	1.97	1.24
EW LT		3.17	1.85	1.10
EW LTw/oT		2.54	1.43	0.68
EW HS		4.05	2.37	1.42
EW HT		4.13	2.42	1.47
EW HTw/oT		3.20	1.80	0.84
EW SHS		5.10	2.94	1.79
EW SHT		5.20	3.01	1.84
EW SHTw/oT		4.26	2.38	1.22
UW SHS		7.50	4.20	2.07
DW LS		7.89	4.43	2.10
DW LT		7.70	4.31	1.94
DW LTw/oT		6.49	3.50	1.14
DW HS		9.36	5.25	2.39
DW HT		9.70	5.49	2.61
DW HTw/oT		7.90	4.28	1.41

TABLE 7 – WEIGHTS & VOLUMES OF BRIDGES

Key to Symbols

STD	= Standard
EW	= Extra Wide
UW	= Ultra Wide
DW	= Double Wide
LS	= Light Steel Decking
LT	= Light Timber Decking
LTw/oT	= Light Timber Decking excluding Timber components
HS	= Heavy Steel Decking
HT	= Heavy Timber Decking
HTw/oT	= Heavy Timber Decking excluding Timber components
SHS	= Super Heavy Steel Decking
SHT	= Super Heavy Timber Decking
SHTw/oT	= Super Heavy Timber Decking excluding Timber components

NOTES

1. The weights and volumes of Two End Bays include End Posts Baseplates Bearings etc.
2. When calculating bridge weights for launching purposes the bridge should be considered as being made up of Internal Bays only.

TABLE 8 – PROPERTIES OF BRIDGES AND COMPONENTS

BRIDGES

CONSTRUCTION	I (ins⁴)	I (cm⁴)	Z (ins³)	Z (cm³)
SS	13600	566070	446	7308
SSR	31300	1302800	906	14846
DS	27200	1132140	892	14617
DSR	62600	2605600	1812	29693
TS	40800	1698200	1338	21925
TSR	93900	3908400	2718	44540
QS	54400	2264290	1784	29234
QSR	125200	5211200	3624	59380
DD	116688	4856900	1912	31331
DDR	249488	10384440	3838	62890
TD	175032	7285350	2868	47000
TDR	374232	15576650	5757	94340
QD	233376	9713800	3824	62664
QDR	498976	20768880	7676	125780

Light Decking

a. American Association of State Highway Officials (AASHO) loading HS20–44.

b. Up to a 14 long ton (14200kg) axle load (with maximum of 7 long ton (7100kg) wheel) at not less than 5ft (1·52m) spacing.

Heavy Decking

a. British Standard (BS) 153 Pt. 3A HA loading.

b. Up to 45 Units of Type HB loading to BS153 Pt. 3A.

Super Heavy Decking

a. Extra Wide Decking

1. Up to 50 ton (50,802kg) axle (plus additional 33% Impact factor).
2. Four wheels/axle 11ft 6in (3·5m) outside of tyres to 3ft 10in (1·17m) inside of tyres (tyre pressure not exceeding 70 psi).
3. The number of trusses should be such that individual transom seat loading does not exceed 14 tons (14224 kg.).

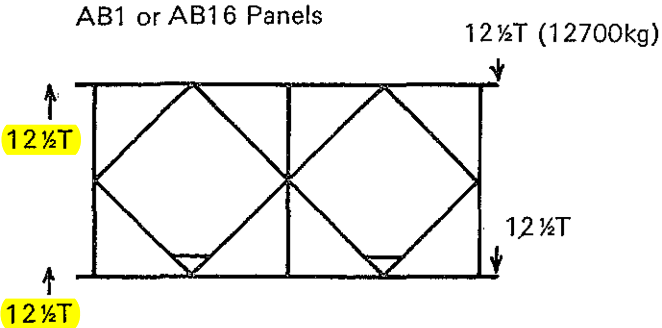
b. Ultra wide decking

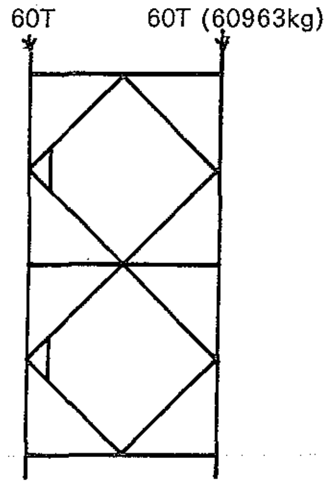
1. Up to 60 ton (60,963kg) axle (plus additional 33% Impact Factor).
2. Two wheels/axle, 8ft 9in (2·67m) centre-to-centre of tyres (tyre pressure not exceeding 55 psi).
3. The number of trusses should be such that individual transom seat loading does not exceed 14 tons (14224 kg.).

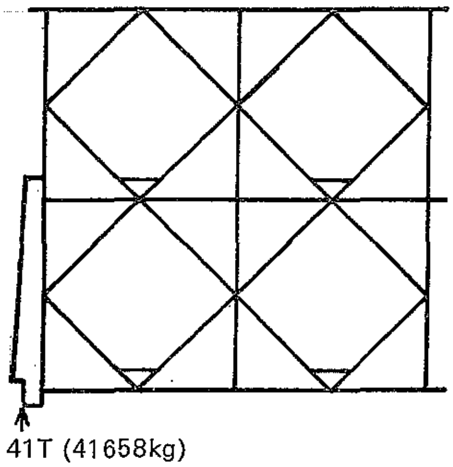
Moment of Inertia of Panel = Ixx = 6800in^4 = 282,035cm^4
Section Modulus of Panel = Zxx = 223in^3 = 3654cm^3
Permissible Bending Moment under static conditions = 330 long tons feet (1002 kNm).
Permissible Bending Moment under dynamic conditions = 245 long tons feet (744kNm).
Permissible stresses on Transoms and Deck Units = 14·2 tons/in^2.

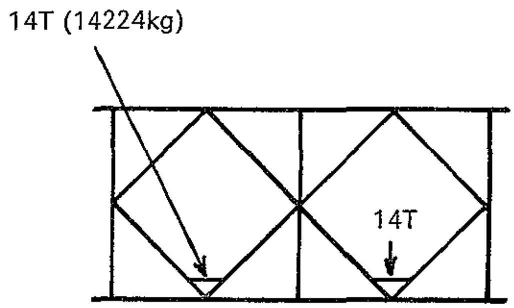
TABLE 8 – Continued

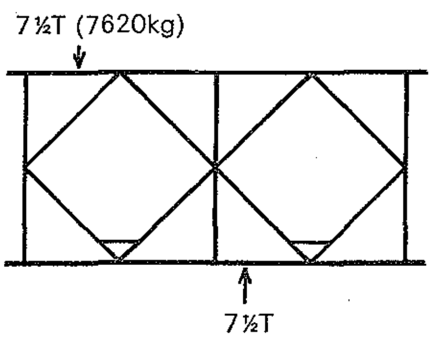
COMPONENTS
AB1 or AB16 Panels

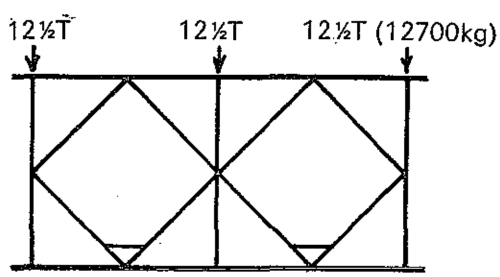












TABLES 9, 10, 11, 12, 13 & 14 –
SHEAR AND BENDING MOMENT CAPACITIES

Tables 9, 10, 11, 12, 13 and 14 are based on the following permissible bending stress and shear capacities:

a) Permissible Bending Stress fb = 13·2 tons/in² (2079 kg/cm²)

b) 1. Permissible Shear Capacity single storey/truss = 25 tons (25400 kg)
   2. Permissible Shear Capacity double storey/truss = 41 tons (41658 kg)

Where fatigue is not a criterion of design (e.g. in formwork design and bridges for temporary use where the number of maximum load cycles is low) the permissible bending stress may be increased to 17·9 tons/in² (2819 kg/cm²). In this case the available bending moment for live load given in Tables 9, 10, 11, 12, 13 and 14 may be increased by the following amounts:

Construction	*Increased Available LLBM (tons feet)*
SS	176
SSR	354
DS	349
DSR	710
TS	523
TSR	1062
QS	697
QSR	1415
DD	748
DDR	1500
TD	1123
TDR	2250
QD	1495
QDR	3005

Shear capacities can NOT be increased above the value given in the Tables.

TABLE 9 – ACROW PANEL BRIDGE – STANDARD WIDTH

LIGHT STEEL DECKING

TABLE OF SHEAR CAPACITY AVAILABLE FOR LIVE LOAD (TONS OF 2,240 LB)

SPAN FT.	M.	SS	SSR	DS	DSR	TS	TSR	QS	QSR	DD	DDR	TD	TDR	QD	QDR
10	3·05	48	48	98	98	148	147	197	196	161	161	243	242	324	323
20	6·10	47	47	96	96	146	144	195	193	159	158	240	239	320	319
30	9·15	46	45	95	94	144	142	193	191	157	156	237	236	317	315
40	12·20	45	44	93	92	142	140	191	188	155	154	235	232	314	311
50	15·25	44	42	92	90	141	138	189	185	153	151	232	229	311	307
60	18·30	43	41	90	88	139	136	187	182	151	149	230	226	308	303
70	21·35	42	40	89	86	137	133	185	179	149	147	227	223	305	300
80	24·40	41	38	87	84	136	131	183	176	147	144	225	220	302	296
90	27·45	39	37	86	82	134	128	181	173	146	142	222	217	299	292
100	30·50	38	36	85	81	132	126	179	171	145	139	219	213	295	287
110	33·35	37	35	83	79	129	122	177	168	142	137	217	210	292	283
120	36·60	36	33	82	78	127	120	175	165	140	135	215	207	289	279
130	39·65	35	32	80	76	125	118	173	162	138	133	212	204	286	275
140	42·70	34	30	79	74	123	115	171	159	136	130	209	201	283	271
150	45·75	33	29	77	73	121	113	169	156	134	128	207	198	280	267
160	48·80	32	28	76	71	119	110	167	153	132	125	204	195	277	263
170	51·85	31	26	74	69	117	108	165	150	130	123	202	192	274	259
180	54·90	30	25	73	67	115	106	163	147	128	120	199	188	270	255
190	57·95	28	24	71	65	113	104	161	144	126	117	196	184	266	251
200	61·00	27	23	70	62	111	102	159	142	124	114	192	180	262	246

A

TABLE 9 – ACROW PANEL BRIDGE – STANDARD WIDTH

LIGHT STEEL DECKING

TABLE OF BENDING MOMENT AVAILABLE FOR LIVE LOAD (TONS (2,240 LB) FEET)

SPAN FT.	M.	SS	SSR	DS	DSR	TS	TSR	QS	QSR	DD	DDR	TD	TDR	QD	QDR
10	3·05	492	—	—	—	—	—	—	—	—	—	—	—	—	—
20	6·10	483	—	—	—	—	—	—	—	—	—	—	—	—	—
30	9·15	469	969	957	—	—	—	—	—	—	—	—	—	—	—
40	12·20	450	946	932	—	—	—	—	—	—	—	—	—	—	—
50	15·25	424	917	900	1883	1376	—	1852	—	—	—	—	—	—	—
60	18·30	395	881	861	1835	1330	—	1798	—	—	—	—	—	—	—
70	21·35	359	830	815	1776	1274	2714	1732	—	1855	—	—	—	—	—
80	24·40	318	792	764	1708	1212	2630	1658	3566	1782	3834	2742	—	—	—
90	27·45	273	738	704	1633	1139	2532	1575	3428	1697	3733	2633	—	3558	—
100	30·50	223	675	637	1547	1060	2425	1478	3297	1604	3620	2510	5535	3408	—
110	33·55	166	612	565	1455	970	2302	1374	3149	1500	3496	2376	5370	3242	7242
120	36·60	106	538	485	1352	876	2175	1261	2992	1387	3359	2230	5187	3059	7015
130	39·65	—	460	397	1239	768	2032	1137	2820	1261	3214	2068	4990	2863	6767
140	42·70	—	373	306	1120	658	1980	1006	2631	1130	3050	1895	4778	2648	6502
150	45·75	—	282	204	992	535	1715	863	2429	1086	2880	1712	4550	2423	6210
160	48·80	—	—	—	856	408	1540	712	2220	836	2696	1512	4310	2180	5912
170	51·85	—	—	—	707	258	1354	547	1985	671	2502	1300	4045	1918	5580
180	54·90	—	—	—	550	—	1108	378	1750	501	2290	1080	3770	1638	5240
190	57·45	—	—	—	387	—	900	—	1485	318	2075	845	3480	1348	4880
200	61·00	—	—	—	215	—	675	—	1225	—	1865	595	3180	1043	4500

LOADING—HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS— SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.
Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.
(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357
PH. (207) 737-2007 FAX (207) 737-2008

Project: Burke, Vt
Contractor: TBuck Construction Inc

Value Engineering Design: Calderwood Engineering
Design Computations by: Eric T. Calderwood, PE

Project Notes:
Vermont Agency of Transportation project Number BRF-0269(13)
Vermont State Rte 114 over Dishmill Brook
Bridge Number 13

Temporary Bridge using Acrow 300 Single Single unreinforced
Timber Crane Matt Abutments

Design Specification: From Acrow manual for 300 for allowable live loads on system

Timber: NDS for wood construction ASD 2005

Live Load: HS20-44

$\gamma_{wood} := 45\ \text{pcf}$ $\qquad L_{sp} := 50\ \text{ft}$

$\gamma_{soil} := 125\ \text{pcf}$ $\qquad k_0 := 0.5$ *at rest soil pressure coefficient for typical granular backfill*

Check Flexure:

$SS_{50f} := 424 \cdot 2240\ \text{ft}\cdot\text{lbf} = 949.76\ \text{ft}\cdot\text{kip}$ *Capacity of Single-Single Standard width Bridge for live load translated from Long Tonnes - ft to Kip-Ft*

$SS_{add'l} := 176 \cdot 2240\ \text{ft}\cdot\text{lbf} = 394.24\ \text{ft}\cdot\text{kip}$ *Addl Flexural Capacity due to temporary application*

$SS_{moment} := SS_{50f} + SS_{add'l} = 1344\ \text{ft}\cdot\text{kip}$

$IM := min\left(\dfrac{50\ \text{ft}}{L_{sp} + 125\ \text{ft}}, 30\%\right) = 28.57\%$ *Impact 3.8.2 AASHTO Std Specifications 17th edition*

$LL_{mom} := 627.9\ \text{ft}\cdot\text{kip} \cdot (100\% + IM) = 807.3\ \text{ft}\cdot\text{kip}$ *Capacity of Bridge is > demand therefor Acrow 300 Single-Single standard width w/ light steel decking is okay for this application*

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Check Shear:

$$SS_{50v} := 44 \cdot 2240 \ \text{lbf} = 98.56 \ \text{kip}$$

Capacity of Single-Single Standard width Bridge for live load translated from Long Tonnes to Kips (shear Cap)

$$LL_{shear} := 58.5 \ \text{kip} \cdot (100\% + IM) = 75.21 \ \text{kip}$$

Capacity of Bridge is > demand therefor Acrow 300 Single-Single standard width w/ light steel decking is okay for this application

Check Soil Bearing Capacity & Abutment timber footing/ distribution beams:

$$Acrow_{dead} := \frac{((1) \cdot 2.30 + (3) \cdot 0.58 + (1) \cdot 2.76 + (3) \cdot 1.62) \cdot 2240 \ \text{lbf}}{2} = 13.0592 \ \text{kip}$$

$$LL_{React} := \frac{LL_{shear}}{(100\% + IM)} = 58.5 \ \text{kip}$$

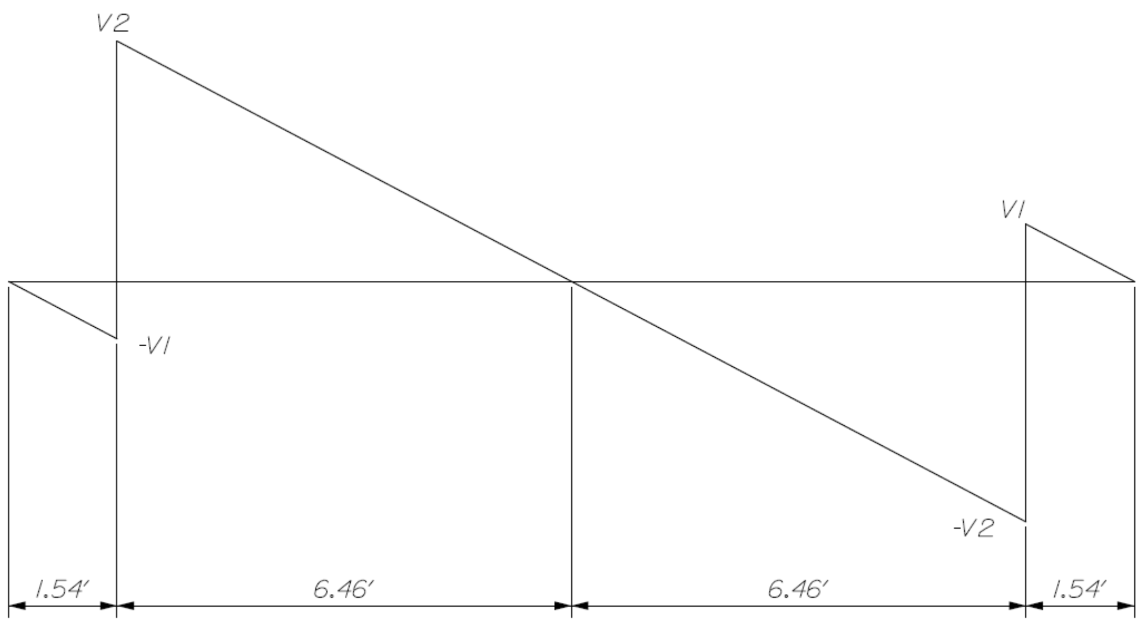
$$A_{width} := 4 \ \text{ft} \qquad t_{abut} := 12 \ \text{in}$$

$$A_{length} := 16 \ \text{ft}$$

$$BP_{abut} := \frac{Acrow_{dead} + LL_{React} + A_{width} \cdot A_{length} \cdot t_{abut} \cdot \gamma_{wood}}{A_{width} \cdot A_{length}} = 1163.1125 \ \text{psf}$$

This is pretty much okay on just about any soil type

$Brg_w := (6\ ft + 5.5\ in) \cdot 2 = 12.9167\ ft$



SHEAR DIAGRAM FOR ABUTMENT LOADS

$V1 := 1.54\ ft \cdot (-BP_{abut}) \cdot A_{width} = -7164.773\ lbf$

$$V2 := \frac{BP_{abut} \cdot A_{width} \cdot A_{length}}{2} + V1 = 30054.827\ lbf$$

$$M_{abut} := \frac{V1}{2} \cdot 1.54\ ft + \frac{V2}{2} \cdot 6.46\ ft = 1098722.592\ in \cdot lbf$$

$$S_{abut} := \frac{A_{width} \cdot t_{abut}^{\ 2}}{6} = 1152\ in^3$$

$$f_{babut} := \frac{M_{abut}}{S_{abut}} = 953.75\ psi$$

$F_b := 625\ psi$

$C_d := 1.15$

$F'_b := F_b \cdot C_d = 718.75\ psi$

$$\frac{f_{babut}}{F'_b} - 1 = 32.696\%$$

Allow a 33% overstress in the timber bending stresses as we are not concerned about some potential damage to the crane matts.

$$\omega D_{beam} := \frac{Acrow_{dead} + LL_{shear}}{(2) \cdot A_{width}} = 11.0342 \ \text{klf}$$

$$M_{distbeam} := \frac{{A_{width}}^2}{2 \cdot 4} \cdot \omega D_{beam} = 264.82 \ \text{in} \cdot \text{kip}$$

$$S_{xxDB} := 66.8 \ \text{in}^3$$

$$f_b := \frac{M_{distbeam}}{S_{xxDB}} = 3.96 \ \text{ksi}$$

Beam is 4 feet long with 2ft unbraced and a 12" wide flange say okay by inspection

Check Backwall:

$$BW_t := 8 \ \text{in}$$

$$S_{bw} := \frac{{BW_t}^2 \cdot 12 \ \frac{\text{in}}{\text{ft}}}{6} = 128 \ \frac{\text{in}^3}{\text{ft}}$$

$$LLS := 2 \ \text{ft}$$

Live Load Surcharge

$$BW_{ht} := 3 \ \text{ft}$$

$$P_{base} := \left(BW_{ht} + LLS\right) \cdot k_0 \cdot \gamma_{soil} = 312.5 \ \text{psf}$$

$$V1_{bw} := 1.54 \ \text{ft} \cdot \left(-P_{base}\right) \cdot 12 \ \frac{\text{in}}{\text{ft}} = -481.25 \ \frac{\text{lbf}}{\text{ft}}$$

$$V2_{bw} := \frac{P_{base} \cdot A_{length} \cdot 12 \ \frac{\text{in}}{\text{ft}}}{2} + V1_{bw} = 2018.75 \ \frac{\text{lbf}}{\text{ft}}$$

$$M_{bw} := \frac{V1_{bw}}{2} \cdot 1.54 \ \text{ft} + \frac{V2_{bw}}{2} \cdot 6.46 \ \text{ft} = 73800 \ \frac{\text{in} \cdot \text{lbf}}{\text{ft}}$$

$$f_{bbw} := \frac{M_{bw}}{S_{bw}} = 576.56 \ \text{psi}$$

$$F_{bbw} := 750 \ \text{psi}$$

$$F'_{bbw} := F_{bbw} \cdot C_d = 862.5 \ \text{psi}$$

8x Eastern Hemlock okay for backwall

Temporary Pedestrian Bridge - Rte 114 over Dishmill Brook Supporting Calculations

In the Town of

Burke

032-br-15

Prepared for:

TBuck Construction Inc

By:

Calderwood Engineering etc

April, 2015

Photo: Newry Maine, Branch Road over Paine Brook
Courtesy of TBuck Construction, Inc & Calderwood Engineering

CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357
PH. (207) 737-2007 FAX (207) 737-2008

Project: Burke, Vt.
Contractor: T Buck Construction Inc.

Temp. Bridge Engineering Design: Calderwood Engineering
Design Computations by: Eric T. Calderwood, PE

Project Notes:
VTrans Specifications as required

Temporary Pedestrian Bridge using steel ASTM A7 Post 1934 ie yield strength = 33 ksi for girders
Steel bolts - Astm A325 7/8" diameter length as required
threadrods - Astm A193 grade B7 (same properties as ASTM A325 bolts)

Design Specification: AASHTO Standard Specifications for Bridge Design 17th edition, 2002 - Allowable stress design to be used for steel construction and concrete abutment footings.

Timber: NDS for wood construction ASD 2005 - APA panel design guide for plywood & OSB

Live Load: Ped Load per AASHTO Standard Specifications 17th edition

$\gamma_{cn} := 150\ pcf$ $\quad$ $\gamma_{soil} := 120\ pcf$ *Saturated Backfill density*

$\gamma_{pvt} := 140\ pcf$ $\quad$ $k_0 := 0.5$ *At Rest lateral soil pressure*

$\gamma_{wood} := 45\ pcf$ $\quad$ $H_{LLS} := 0.0\ ft$ *Height of Live Load Surcharge (none req'd for Ped Loading)*

$E_b := 29000\ ksi$ *Modulus of elasticity of steel* $\quad$ $E_{spf2} := 1400000\ psi$

$F_{yb} := 33\ ksi$ *Steel beam yield* $\quad$ $E_{sr} := 29000\ ksi$ *Modulus of Elasticity of Rebar*

$F_{ys} := 36\ ksi$ *Miscellaneous steel yield*

$F_{yr} := 60\ ksi$ *reinforcing steel yield*

$t_d := 8\ in$ *reinforcing steel yield* $\quad$ $f'_{cb} := 3000\ psi$ *Concrete compression strength for abutment blocks*

$p_{swdl} := 20\ psf$ *sidewalk flooring dead load*

$L_{sp} := 65\ ft$ *Bearing to Bearing span length*

$S_{width} := 5\ ft$ *sidewalk width*

$D_{width} := 10\ ft$ *Maximum deck width for timber matts*

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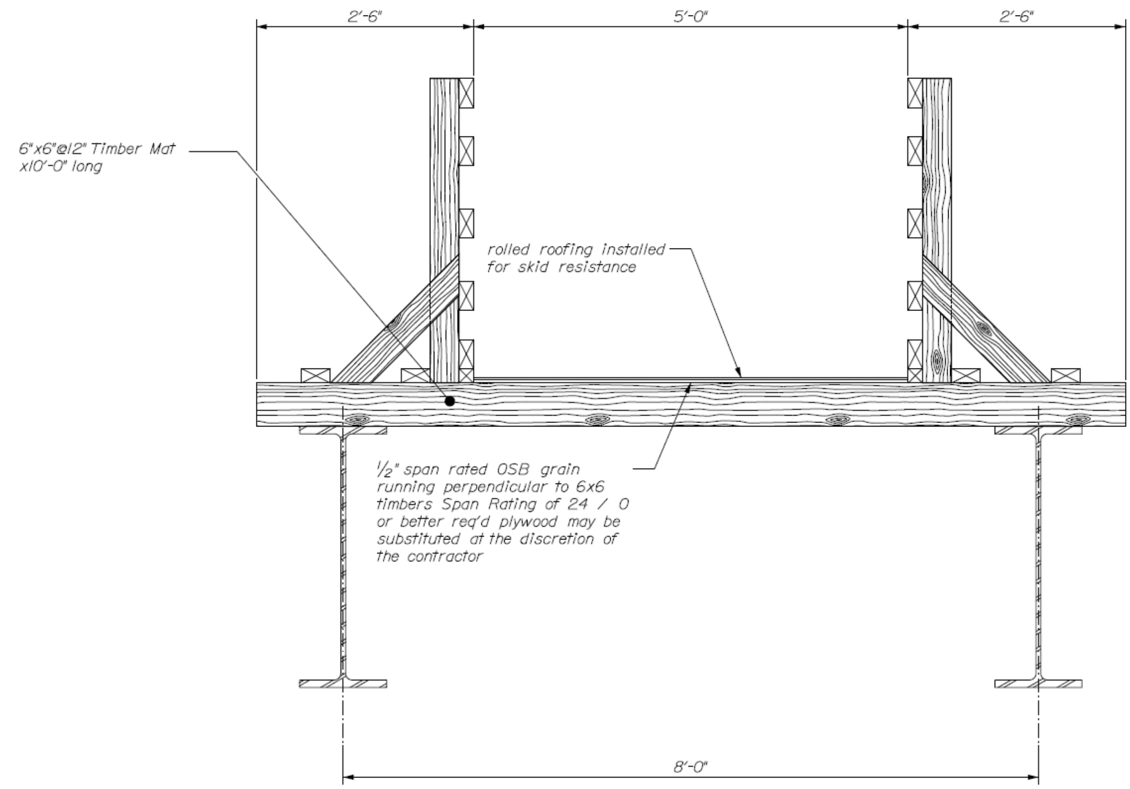

65' Span Ped Bridge Typical Section
1" = 1'-0"

$S_{ave} := 8.0 \text{ ft}$

W36x160 sidewalk beam along with misc additional channels & nailer blocks etc (the 15%)

$W_{swb} := 115\% \cdot 160 \text{ plf} = 184 \text{ plf}$

$$W_{swd} := \frac{p_{swdl} \cdot D_{width}}{2} = 100 \text{ plf}$$

Sidewalk dead weight on sidewalk beam

$W_{pedrail} := 25 \text{ plf}$

Sidewalk Ped Railing applied 100% to sidewalk beam

$P_{swLL} := 60 \text{ psf}$

3.14.1.1

$$W_{swLL} := \frac{S_{width} \cdot P_{swLL}}{2} = 150 \text{ plf}$$

Sidewalk Live Load on SW beam

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Flexural Design Sidewalk Beam

$S_{xcsw} := 542\ in^3$ $\qquad I_{xbsw} := 9750\ in^4$

$t_{fbsw} := 1.02\ in$ $\qquad L_{unbsw} := 13\ ft$

$b_{fbsw} := 12.00\ in$ $\qquad C_{bsw} := 1.0$

$t_{wbsw} := 0.65\ in$

$d_{bsw} := 36.01\ in$

$$J_{bsw} := \frac{2 \cdot \left(t_{fbsw}^{\ 3} \cdot b_{fbsw}\right) + \left(d_{bsw} - 2 \cdot t_{fbsw}\right) \cdot t_{wbsw}^{\ 3}}{3} = 11.5993\ in^4$$

$$I_{ycsw} := \frac{b_{fbsw}^{\ 3} \cdot t_{fbsw}}{12} = 146.88\ in^4$$

$$M_{llsw} := \frac{W_{swLL} \cdot L_{sp}^{\ 2}}{8} = 950.625\ in \cdot kip$$

$$M_{dlsw} := \frac{\left(W_{swb} + W_{swd} + W_{pedrail}\right) \cdot L_{sp}^{\ 2}}{8} = 1958.29\ in \cdot kip$$

$$f_{bSW} := \frac{M_{llsw} + M_{dlsw}}{S_{xcsw}} = 5.367\ ksi$$

$$F_{bSW} := min\left(\frac{50000000\ psi \cdot C_{bsw}}{S_{xcsw}} \cdot \left(\frac{I_{ycsw}}{L_{unbsw}}\right) \cdot \sqrt{0.722 \cdot \left(\frac{J_{bsw}}{I_{ycsw}}\right) + 9.87 \cdot \left(\frac{d_{bsw}}{L_{unbsw}}\right)^2}\ , 0.55 \cdot F_{yb}\right) = 18.15\ ksi$$

$$\Delta_{LLsw} := \frac{5 \cdot W_{swLL} \cdot L_{sp}^{\ 4}}{384 \cdot E_b \cdot I_{xbsw}} = 0.2131\ in$$

Allowable stress from Table 10.3.2.1A
Allowable flexural stress >> actual flexural stress therefor okay

LL deflection of sidewalk beam ~ L/3660 << L/1000 (10.6.2)

Shear Design Sidewalk Beam

$$f_{vsw} := \frac{\left(W_{swLL} + W_{swb} + W_{swd} + W_{pedrail}\right) \cdot L_{sp}}{2 \cdot \left(d_{bsw} - 2 \cdot t_{fbsw}\right) \cdot t_{wbsw}} = 0.68\ ksi$$

$F_{vsw} := 0.33\ F_{yb} = 10.89\ ksi$

shear stress in sidewalk beam < shear stress allowed in sidewalk beam therefor okay

Sidewalk Joists Design: Use Full Dimension 6x6 joists SPF #2 or btr spaced at 12" (crane matts from Richmond)

$d := 6\ in$ $\qquad b := 6\ in$

$S_{xj} := \frac{d^2 \cdot b}{6} = 36\ in^3$ $\qquad I_{xj} := \frac{d^3 \cdot b}{12} = 108\ in^4$

$A_j := d \cdot b = 36\ in^2$ $\qquad J_{sp} := S_{ave} = 8\ ft$

$S_{joists} := 12\ in$ $\qquad \frac{d}{b} = 1$

$\omega_{dl} := p_{swdl} \cdot S_{joists} = 20\ plf$

$\omega_{ll} := P_{swLL} \cdot S_{joists} = 60\ plf$

$M_{joist} := \frac{(\omega_{ll} + \omega_{dl}) \cdot J_{sp}^2}{8} = 640\ ft \cdot lbf$

$f_b := \frac{M_{joist}}{S_{xj}} = 213.33\ psi$

$\Delta_{llj} := \frac{5 \cdot \omega_{ll} \cdot J_{sp}^4}{384 \cdot E_{spf2} \cdot I_{xj}} = 0.0366\ in$ $\qquad \frac{J_{sp}}{\Delta_{llj}} = 2625$

deflection is less than L/1000 therefor okay

$F_b := 575\ psi$

$C_r := 1.00$ *members are 6x6 repetetive use factor does not apply*

$C_d := 1.15$ *Use Cd=1.15 for 2 month duration of load 2.3.2 NDS*

$C_t := 1.0$ *Use will be Less than 150 degrees F sustained 2.3.3 NDS*

$C_L := 1.0$ *Depth is the same as the width - no lateral support is required joists will not attempt to roll over*

$C_f := 1.0$ *for 6x6s Table 4D*

$C_i := 1.0$ *no incising*

$C_m := 1.0$ *Table 4D*

$F'_b := F_b \cdot C_r \cdot C_d \cdot C_t \cdot C_L \cdot C_f \cdot C_i \cdot C_m = 661.25\ psi$

661.25 psi > 50 psi therefor okay

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Sidewalk Joists Design: Use
nominal 6x6 joists SPF #2 or btr
Bearing

$$R_{swj} := \frac{(\omega_{ll} + \omega_{dl}) \cdot J_{sp}}{2} = 320 \text{ lbf}$$

$$L_b := b_{fbsw} = 12 \text{ in}$$

$$f_{cperp} := \frac{R_{swj}}{L_b \cdot b} = 4.44 \text{ psi}$$

$C_b := 1.0$ *bearing is Located w/in 3" of the end of the joist (conservative assumption)*

$C_t := 1.0$ *Use will be Less than 150 degrees F sustained 2.3.3 NDS*

$C_m := 0.67$ *Per Table 4D for compression perpendicular to grain*

$F_{cperp} := 335 \text{ psi}$ *Per Table 4D for compression perpendicular to grain for SPF South*

$$F'_{cperp} := F_{cperp} \cdot C_b \cdot C_t \cdot C_m = 224.45 \text{ psi}$$

Allowable Bearing stress is >>>> than applied bearing stress therefor okay

Sidewalk Joists Design: Use
nominal 6x6 joists SPF #2 or btr
Horizontal Shear

$$f_v := \frac{R_{swj} \cdot 3}{2 \cdot b \cdot d} = 13.3333 \text{ psi}$$

by definition horizontal shear stress at NA of rectangular section (ref NDS 3.4.2 eqn 3.4-2) - note shear is calculated at the reaction for this joist which is conservative - shear design section is allowed to be taken at a distance d from the support, but that will result in a lower stress and therefor we are not concerned with it.

$F_v := 135 \text{ psi}$

$C_d := 1.15$ *Use Cd=1.15 for 2 month duration of load 2.3.2 NDS*

$C_t := 1.0$ *Use will be Less than 150 degrees F sustained 2.3.3 NDS*

$C_m := 1.00$ *Per Table 4D for shear parallel with the grain*

$$F'_v := F_v \cdot C_d \cdot C_t \cdot C_m = 155.25 \text{ psi}$$

Allowable Horizontal Shear stress is greater than applied horizontal shear stress therefor okay

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Plywood Decking: use 1/2" OSB or plywood decking (sheathing)

Use APA rated sheathing Exp 1 PS2-04 40/20 OSB 5/8" Design per APA Panel Design Specification - 5/8" formply - used in good condition will be allowed as a substitute for the OSB directly at the contractors option

$$EI_x := 60000\ \frac{lbf \cdot in^2}{ft}$$

All Design Values are from the APA Panel Design Specification Table 4A - OSB 24/0 span rating

$$EI_y := 11000\ \frac{lbf \cdot in^2}{ft}$$

$$F_bS_x := 300\ \frac{in \cdot lbf}{ft}$$

$$F_bS_y := 97\ \frac{in \cdot lbf}{ft}$$

$$F_s := 130\ \frac{lbf}{ft}$$

$$F_{cperp} := 115\ psi$$

Not listed in APA panel Design but from engineering judgement and in comparison with plyform which is 210 psi & 335 psi for SPF therefor using engineering judgement 115 psi seems like a reasonable bearing stress to this engineer

$$C_d := 1.15$$

From APA panel Design section 4.5.1 & 4.5.2

$$C_{mbs} := 0.75$$

$$C_{mE} := 0.85$$

$$C_{mc} := 0.20$$

$$\omega_s := 1.5\ psf$$

Panel Properties from Table 6 for 1/2" panels APA panel design specification

$$t_s := 0.500\ in$$

$$A_s := 6.0\ \frac{in^2}{ft}$$

$$I := 0.125\ \frac{in^4}{ft}$$

$$S := 0.500\ \frac{in^3}{ft}$$

$$Q := 0.375\ \frac{in^3}{ft}$$

Statical Moment

$$RQ := 4.00\ \frac{in^2}{ft}$$

Rolling Shear Constant

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Run the grain parallel to the joist supports in order to preserve as many full size sheets as possible 9ie no need to cut 8 footers for location - orientation grain perpendicular to joists - 3 span condition

$$P_{swLL} = 60\ psf$$

$$M_{bs} := \frac{(P_{swLL} + \omega_s + 5\ psf) \cdot S_{joists}^{\ 2}}{10} = 79.8\ \frac{in \cdot lbf}{ft}$$

$$f_{bs} := \frac{M_{bs}}{S} = 159.6\ psi$$

$$F_b S_x \cdot C_d \cdot C_{mbs} = 258.75\ \frac{in \cdot lbf}{ft}$$

$$\Delta_{sLL} := \frac{P_{swLL} \cdot (S_{joists})^4}{1743 \cdot EI_x} = 0.001\ in$$

$$\frac{S_{joists}}{\Delta_{sLL}} = 12104.1667$$

Deflection is L/12100 << L/1000 therefor okay

$$w_{rs} := \frac{20\ \frac{in}{ft} \cdot F_s}{S_{joists} - b} = 433.3333\ psf$$

$$W_{rsall} := w_{rs} \cdot C_d \cdot C_{mbs} = 373.75\ psf$$

For a one span condition Maximum Allowable uniform load in PSF

$$F_{sstress} := \frac{F_s \cdot C_d \cdot C_{mbs}}{RQ} = 28.0313\ psi$$

Allowable Rolling Shear Stress

$$f_s := \frac{(P_{swLL} + \omega_s + 5\ psf) \cdot (S_{joists} - b)}{20\ \frac{in}{ft} \cdot RQ} = 4.9875\ psi$$

Actual Rolling Shear stress for three span condition

Rolling shear stress is << Allowable Rolling shear stress (Horizontal shear in panels)

$$f_{cperp} := \frac{(P_{swLL} + \omega_s + 5\ psf) \cdot S_{joists}}{12\ \frac{in}{ft} \cdot b} = 0.9236\ psi$$

$$F'_{cperp} := F_{cperp} \cdot C_{mc} = 23\ psi$$

Allowable Bearing stress on panels under wet conditions

Bearing stress << Allowable Bearing stress therefor okay

$$\frac{d \cdot b \cdot \gamma_{wood}}{1\ ft} + \omega_s + 5\ psf = 17.75\ psf$$

total SW floor loading check <20psf therefor okay

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Ped Rail Design

$b_{rail} := 3.5\ in$ $\qquad$ $d_{rail} := 1.5\ in$

$b_{post} := 3.5\ in$ $\qquad$ $d_{post} := 3.5\ in$

$S_{post} := 4\ ft$ $\qquad$ $L_{post} := 42\ in + 0.5\ in = 3.5417\ ft$

$w := 50\ plf$

Each rail is designed for a uniform load of 50plf per rail both vertically and horizontally simultaneousl - Posts are designed for the reaction of the upper most rail only or wL where L is the post spacing per AASHTO Std Spec. 2.7.3.2.2 & 2.7.3.2.3 respectively

$$M_{uy} := \frac{w \cdot S_{post}^{\ 2}}{10} = 960\ in \cdot lbf$$

Use 12' 2x's & 4 ft spacing of posts ie (3 span condition, one span loaded)

$$M_{ux} := \frac{w \cdot S_{post}^{\ 2}}{10} = 960\ in \cdot lbf$$

Use 12' 2x's & 4 ft spacing of posts ie (3 span condition, one span loaded)

$$S_x := \frac{b_{rail}^{\ 2} \cdot d_{rail}}{6} = 3.0625\ in^3 \qquad S_y := \frac{d_{rail}^{\ 2} \cdot b_{rail}}{6} = 1.3125\ in^3$$

$$f_{bx} := \frac{M_{ux}}{S_x} = 313.5\ psi \qquad f_{by} := \frac{M_{uy}}{S_y} = 731.4\ psi$$

$$S_{xpost} := \frac{b_{post}^{\ 2} \cdot d_{post}}{6} = 7.1458\ in^3$$

$F_b := 775\ psi$ *From Table 4a NDS*

$C_r := 1.00$

$C_d := 1.25$ *Use Cd=1.25 for "construction" level duration (7 days sustained)*

$C_t := 1.0$ *Use will be Less than 150 degrees F sustained 2.3.3 NDS*

$C_L := 1.0$ *Ends and compression face are prevented from rotation via plank fastening/ end support blocking NDS 3.3.3.3 mid span blocking is also provided*

$C_f := 1.5$ *for 2x4s Table 4A - 2x6 railings may be used in lieu of 2x4 railings at the contractors option - also Cf for 4x4 post is 1.5 as well*

$C_i := 1.0$ *no incising*

$C_m := 1.0$ $F_b \cdot C_f = 1162.5\ psi$ *<1150 psi therefor Cm= 1.0 (Table 4A)*

$C_{fux} := 1.0$ *not being used flat*

$C_{fuy} := 1.1$

$F'_{by} := F_b \cdot C_r \cdot C_d \cdot C_t \cdot C_L \cdot C_f \cdot C_i \cdot C_m \cdot C_{fuy} = 1598.4375\ psi$

$F'_{bx} := F_b \cdot C_r \cdot C_d \cdot C_t \cdot C_L \cdot C_f \cdot C_i \cdot C_m \cdot C_{fux} = 1453.125\ psi$

$E'_{min} := 400000\ psi$ *From Table 4a NDS*

$l_e := 1.63 \cdot S_{post} + 3 \cdot d_{rail} = 6.895\ ft$ *3.3.3 NDS*

$R_b := \sqrt{\dfrac{l_e \cdot d_{rail}}{{b_{rail}}^2}} = 3.183$ *3.3.3.6 NDS*

$F_{bE} := \dfrac{1.20 \cdot E'_{min}}{{R_b}^2} = 47377.3266\ psi$

$$\frac{f_{bx}}{F'_{bx}} + \frac{f_{by}}{F'_{by} \cdot \left(1 - \left(\frac{f_{bx}}{F_{bE}}\right)^2\right)} = 0.6733$$

<1.0 therefor biaxial bending is okay for each railing NDS 3.9.2 (No compression)

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$M_{post} := w \cdot S_{post} \cdot L_{post} = 8500 \ in \cdot lbf$

$f_{cpost} := \dfrac{w \cdot S_{post}}{b_{post} \cdot d_{post}} = 16.3265 \ psi$

$f_{bpost} := \dfrac{M_{post}}{S_{xpost}} = 1189.5044 \ psi$

$F'_{bx} := F_b \cdot C_r \cdot C_d \cdot C_t \cdot C_L \cdot C_f \cdot C_i \cdot C_m \cdot C_{fux} = 1453.125 \ psi$

$F_c := 1000 \ psi$ *From Table 4a NDS*

$C_{mc} := 0.80$ *From Table 4a NDS*

$C_{fc} := 1.15$ *From Table 4a NDS*

$F_{starC} := F_c \cdot C_d \cdot C_{mc} \cdot C_t \cdot C_{fc} \cdot C_i = 1150 \ psi$ *3.7.1 NDS*

$k_e := 2.1$ *Effective length factor for cantilevered columns per NDS appendix G*

$l_{epost} := k_e \cdot L_{post} = 7.4375 \ ft$

$F_{cE} := \dfrac{0.822 \cdot E'_{min}}{\left(\dfrac{l_{epost}}{d_{post}}\right)^2} = 505.6517 \ psi$ *3.7.1 NDS*

$c := 0.8$ *3.7.1 NDS (for sawn lumber)*

$C_p := \dfrac{1 + \dfrac{F_{cE}}{F_{starC}}}{2 \cdot c} - \sqrt{\left(\dfrac{1 + \dfrac{F_{cE}}{F_{starC}}}{2 \cdot c}\right)^2 - \dfrac{\dfrac{F_{cE}}{F_{starC}}}{c}} = 0.3899$ *3.7.1 NDS*

$F'_c := F_{starC} \cdot C_p = 448.3523 \ psi$ *3.7.1 NDS*

$\left(\dfrac{f_{cpost}}{F'_c}\right)^2 + \dfrac{f_{bpost}}{F'_{bx} \cdot \left(1 - \dfrac{f_{cpost}}{F_{cE}}\right)} = 0.8472$ *3.9.2 NDS <1.0 therefor - combined flexure & axial stress okay for 4x4 posts at 4' spacing*

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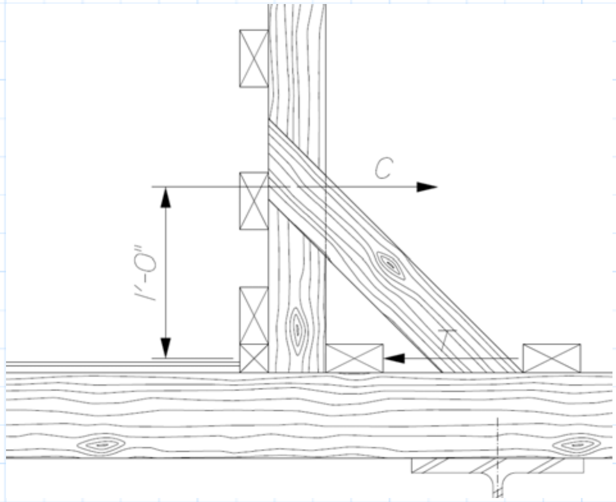
Check Kicker for compression due to railing design loads



$$P := w \cdot S_{post} = 200 \ lbf$$

$$C := \frac{P \cdot (L_{post} - 0.75 \ in)}{12 \ in} = 695.8333 \ lbf$$

$$T := \frac{P \cdot (L_{post} - 12.75 \ in)}{12 \ in} = 495.8333 \ lbf$$

Reaction of 2x2 is carried to OSB and is transmitted to the 6x6 decking through shear by inspection no further analysis is required

$$C_{kicker} := \frac{C}{\sin(45 \ deg)} = 984.0569 \ lbf$$

$$f'_{ckick} := \frac{C_{kicker}}{3.5 \ in \cdot 1.5 \ in} = 187.4394 \ psi$$

$F_c = 1000 \ psi$ $F_{starC} = 1150 \ psi$

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$k_e := 1.0$ *Effective length factor for pinned pinned columns per NDS appendix G*

$$l_{ekick} := k_e \cdot \frac{12\ in}{\sin(45\ deg)} = 1.4142\ ft$$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_{ekick}}{1.5\ in}\right)^2} = 2568.75\ psi$$ *3.7.1 NDS*

$c := 0.8$ *3.7.1 NDS (for sawn lumber)*

$$C_p := \frac{1 + \frac{F_{cE}}{F_{starC}}}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_{starC}}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{F_{starC}}}{c}} = 0.8842$$ *3.7.1 NDS*

$$F'_c := F_{starC} \cdot C_p = 1016.7729\ psi$$ *3.7.1 NDS* *Kicker Capacity okay as f'c<<F'c therefor okay*

Check Nailing Capacity

$Z_{16d} := 120\ lbf$ *Table 11n* $C_d = 1.25$

$$N_{req'd} := \frac{C_{kicker}}{Z_{16d} \cdot C_d} = 6.5604$$ *Use (2) kickers and use (4) 16d nails in each*

$$R_{abut} := \frac{\left(W_{swLL} + W_{swb} + W_{swd} + W_{pedrail}\right) \cdot L_{sp}}{2} = 14.92 \ kip$$

$$W_{abut} := 2 \ ft$$

$$L_{abut} := 8 \ ft$$

$$A_{brgpressure} := \frac{R_{abut} \cdot 2}{W_{abut} \cdot L_{abut}} = 1864.69 \ psf$$

$$M_{abut} := \frac{A_{brgpressure} \cdot W_{abut} \cdot S_{ave}^{\ 2}}{8} = 358.02 \ in \cdot kip$$

$$A_{stt} := 0.31 \ in^2 \cdot 4 = 1.24 \ in^2$$

$$f'_c := 3000 \ psi$$

$$E_c := 57000 \cdot \sqrt{f'_c \cdot psi} = 3122018.58 \ psi \qquad n := \frac{E_{sr}}{E_c} = 9.2889$$

$$C_{rebar} := 1.5 \ in \qquad \varphi_{rebar} := 0.625 \ in$$

$$t_{conc} := 16 \ in$$

Cracked Neutral Axis Flexure:

$A_{strans} := A_{stt} \cdot n = 11.5182\ in^2$ *area of steel transformed to concrete*

$c_{cr} := 3.2414\ in$ *distance to cracked neutral axis (assumed)*

$y_{sb} := C_{rebar} + \frac{\varphi_{rebar}}{2} = 1.8125\ in$

$y_{compb} := t_{conc} - \frac{c_{cr}}{2} = 14.3793\ in$

$A_{compbl} := c_{cr} \cdot W_{abut} = 77.7936\ in^2$

$y_{bar} := \frac{A_{compbl} \cdot y_{compb} + A_{strans} \cdot y_{sb}}{A_{compbl} + A_{strans}} = 12.7586\ in$

$c_{cr} := t_{conc} - y_{bar} = 3.2414\ in$

$I_{cr} := \frac{\left(c_{cr}^{\ 3} \cdot W_{abut}\right)}{12} + A_{compbl} \cdot \left(y_{compb} - y_{bar}\right)^2 + A_{strans} \cdot \left(y_{sb} - y_{bar}\right)^2 = 1652.53\ in^4$

$S_{crstl} := \frac{I_{cr}}{\left(y_{bar} - y_{sb}\right) \cdot n} = 16.25\ in^3$ $S_{crconc} := \frac{I_{cr}}{t_{conc} - y_{bar}} = 509.82\ in^3$

$f_{ss} := \frac{M_{abut}}{S_{crstl}} = 22.0283\ ksi$ $F_t := 24\ ksi$ *8.15.2.2 for Grade 60 rebar*

$f_{conc} := \frac{M_{abut}}{S_{crconc}} = 0.7022\ ksi$ $F_c := 0.4 \cdot f'_c = 1.2\ ksi$ *8.15.2.1.1 for 3000 psi concrete*

Tensile stress in the rebar is < 24 ksi & Compressive stress in the concrete is < 1200 psi therefor okay for 16" deep abutment pad & (4) # 5 bars top & bottom use #5 bars transverse at 18" spacing max